Water Resources

Alberta

ENVIRONMENT

Willow Creek Project
Pine Coulee Reservoir

Geotechnical Design Report - Level I

November 1985

Prepared by: Shiu F. Jung, P. Eng.
J. E. McClung, P. Eng.
K. Tedder, P. Geol.

Reviewed by: N. Weimer, P. Eng.

Willow Creek Project

Pine Coulee Reservoir

Geotechnical Design Report - Level I

November 1985

Location:	Secs. 32, 33 & 34 of Tp. 13, Rg. 28, W4 Secs. 3, 10, 15, 22, 28 & 33 of Tp. 14, Rg. 28, W4 Sec. 3 of Tp. 15, Rg. 28, W4
File No.:	D 21399 A

Geotechnical Branch
Design and Construction Division
Alberta Environment

Table of Contents

Page

1. Introduction 1

2. Site Investigation and Testing 2

3. General Geology 9

    3.1 Bedrock Geology 10
    3.2 Bedrock Topography 11
    3.3 Surficial Geology 12
    3.4 Geomorphic Development 17
    3.5 Hydrology 21

4. Engineering Geology 23

    4.1 Diversion Weir Site 23
    4.2 Diversion Channel 25
    4.3 Damsite 27
    4.4 Causeway at Highway #527 31

5. Damsite Selection 33

6. Diversion Weir Site Selection 34

7. Reservoir Integrity 35

    7.1 Reservoir Watertightness 35
    7.2 Reservoir Slope Stability 36

8. Foundation Conditions 37

    8.1 Diversion Weir 37
    8.2 Diversion Channel 38
    8.3 Reservoir Inlet Structure 40
    8.4 Saddle Dykes 41
    8.5 Dam Embankment 42
    8.6 Causeway @ Highway #527 43

Table of Contents - (cont'd)

Page

9. **Design Considerations** 44

9.1 Diversion Weir 44
9.2 Diversion Channel 47
9.3 Reservoir Inlet Structure 48
9.4 Saddle Dykes 49
9.5 Dam Embankment Design 50

9.5.1 General 50
9.5.2 Slope Stability Analysis 50

a) Stability of the Dam Embankment Slopes During Construction

b) Stability of the Dam Embankment Upstream Slope During Drawdown

c) Long Term Stability of the Dam Embankment Downstream Slope

d) Stability of the Natural Dam Embankment Slopes

9.5.3 Dam Embankment Settlement 55
9.5.4 Wick Drains 56
9.5.5 Seepage Considerations 56
9.5.6 Scour Protection 57
9.5.7 Impervious Borrow Materials 58
9.5.8 Instrumentation 59
9.5.9 Low Level Outlet Structure 60

a) General
b) Conduit Settlement

Table of Contents - (cont'd)

Page

9.6 Causeway @ Highway #527 62

9.6.1 General 62
9.6.2 Slope Stability Analysis 62

a) Stability of the Causeway Slope During Construction

b) Stability of the Causeway Slopes During Drawdown

9.6.3 Wick Drains 66
9.6.4 Scour Protection 66
9.6.5 Impervious Borrow Materials 67

9.7 East-West Road Causeway @ Northern Boundary of 33-14-28-w4 68

10. Availability of Granular Materials & Rock Riprap 69

11. Conclusions & Recommendations 69

12. Additional Study Requirements for Final Design 72

13. Cost Estimate 74

TABLES

1. Pine Coulee Site Investigation Data
2. Simplified Sequence of Glacial Events in Southern Alberta
3. Soil Strength Parameters Used for Design
4. Summary of Consolidation Tests

LIST OF DRAWINGS

1. Surficial Geology
2. Borehole and Stratigraphic Sections Location Plan
3. Borehole and Stratigraphic Section J-J' Location Plan
4. Borehole Location Plan
5. Bedrock Topography of Dam Site Area
6. Stratigraphic Section A-A' (~40 m D/S of Pine Coulee Dam Site ℄)
7. Stratigraphic Section B-B' (~ 60 m U/S of Pine Coulee Dam Site ℄)
8. Stratigraphic Section C-C' (~250 m U/S of Pine Coulee Dam Site ℄)
9. Stratigraphic Section D-D' (~350 m U/S of Pine Coulee Dam Site ℄)
10. Stratigraphic Section E-E' Northwest (Right) Abutment
11. Stratigraphic Section F-F' Northwest Side of Valley Bottom
12. Stratigraphic Section G-G' Southeast Side of Valley Bottom
13. Stratigraphic Section H-H' (~25 m U/S of Willow Creek Weir Site)
14. Stratigraphic Section I-I' (~ 80 m D/S of Willow Creek Weir Site)
15. Stratigraphic Section J-J' (~ 40 m North of Hwy. #527)
16. General Plan
17. Site Plan
18. Diversion Weir Plan & Sections
19. Diversion Weir Structure
20. Diversion Channel, Headgates Structure
21. Diversion Channel Stratigraphic Profile, Sta. 0+000 to Sta. 1+800
22. Diversion Channel Stratigraphic Profile, Sta. 1+800 to Sta. 3+550
23. Reservoir Inlet, Plan and Profile
24. Reservoir Inlet Chute, Stability Analysis, Drawdown Condition
25. Saddle Dyke
26. Saddle Dyke Stability Analysis, End of Construction, U/S Slope
27. Saddle Dyke Stability Analysis, Drawdown Condition, U/S Slope
28. Saddle Dyke Stability Analysis, End of Construction, D/S Slope
29. Saddle Dyke Stability Analysis, Steady State Seepage D/S Slope

LIST OF DRAWINGS - (cont'd)

30. Dam Embankment, General Plan

31. Typical Dam Embankment Cross Section

32. Outlet Works

33. Dam Embankment Stratigraphic Section

34. Dam Embankment, Cross Section and Strength Parameters Used for Effective Stress Analysis

35. Dam Embankment (Stage I) Slope Stability Analysis (Total Stress)

36. Dam Embankment (Stage II) Slope Stability Analysis (Total Stress)

37. Existing Reservoir Bank, Slope Stability Analysis - Reservoir Drawdown

38. Causeway @ Highway #527

39. Stratigraphic Section @ 40 m North of Hwy. # 527

40. Causeway @ Hwy. #527, Cross Section & Strength Parameters Used for Effective Stress Analysis

41. Causeway @ Hwy. #527, Slope Stability Analysis (Stage I) Total Stress

42. Causeway @ Hwy. #527, Slope Stability Analysis (Stage II) Total Stress

43. Causeway E-W Road, N. Bdy. of 33-14-28-W4

1.0 Introduction

Due to increased water demands and continued drought conditions in the southern part of the province, additional water storage within the Willow Creek basin appears to be imminent. As a result, a preliminary design and feasibility assessment together with a cost estimate for an off-stream storage reservoir within Pine Coulee along with the associated diversion works from Willow Creek was requested by Alberta Environment Planning Division.

The major components associated with the development of this storage reservoir include the construction of an earth dam across the downstream end of Pine Coulee, a saddle dyke along the southeast periphery of the proposed reservoir, a major causeway at the Highway #527 coulee crossing and a minor causeway crossing near the northern boundary of the reservoir. In addition, the major components associated with the diversion works from Willow Creek include the construction of a diversion weir on Willow Creek, a diversion headgate structure, a diversion channel and the Pine Coulee reservoir inlet structure. The general layout of all the major components is shown on Drawing No. 16.

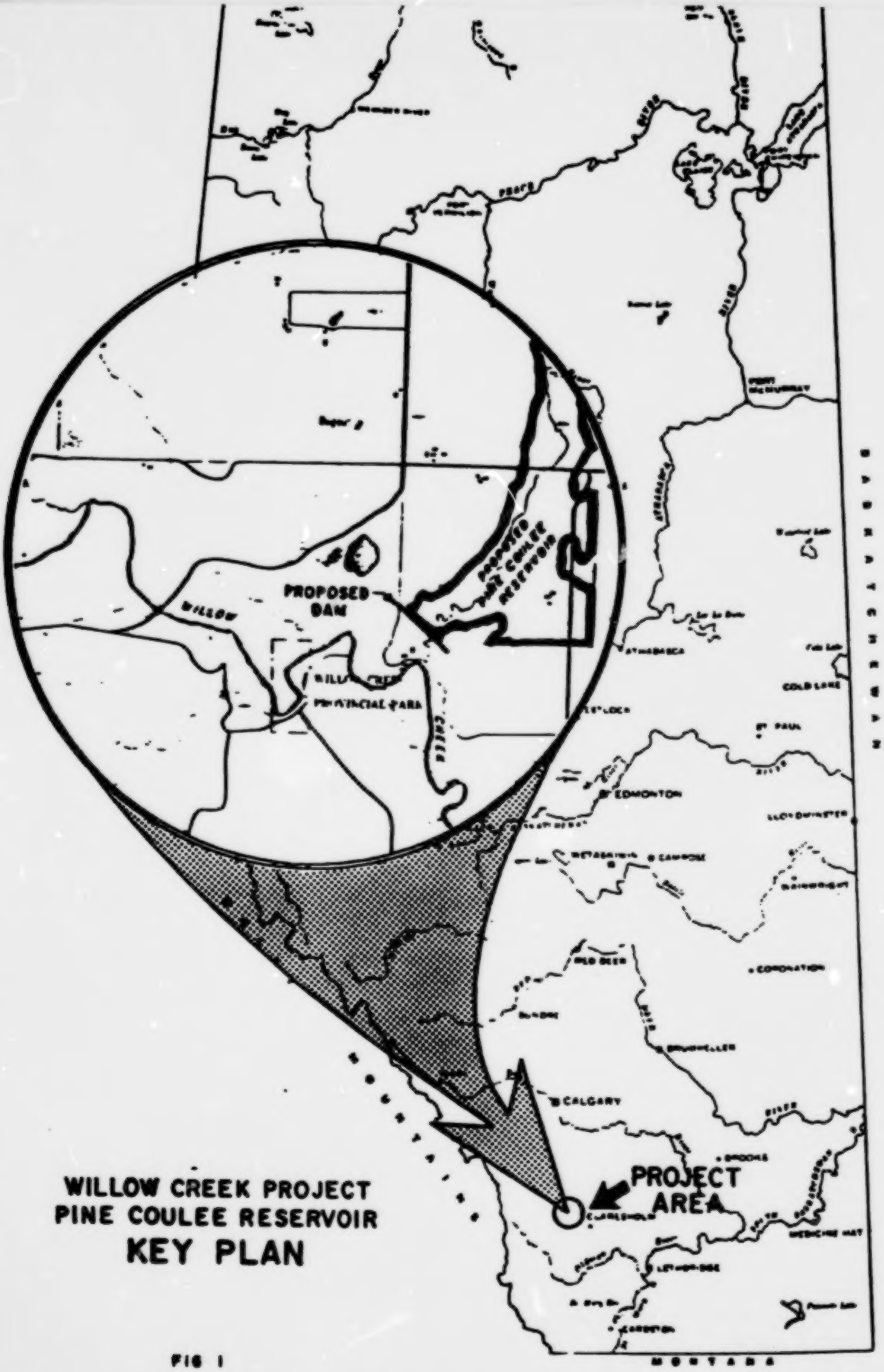
The proposed Pine Coulee dam is approximately 21 m in height and 450 m in length along its crest. It would be located about 13 km southwest of the Town of Stavely within the SW 34-13-28-W4, approximately 400 m upstream of the confluence of Pine Coulee and

Willow Creek (Fig. 1). The resulting reservoir would be approximately 12 km in length and would cover some 6.0 km^2 in area. The storage capacity of the reservoir would be approximately 50.3 million m^3 at the design FSL elevation of 1052.5 and a minimum ponding elevation of 1044 was assumed for design purposes.

The reservoir created by the construction of the dam will flood four local roads which presently cross the coulee. For cost estimating purposes, it was assumed that the most southerly road would be re-routed across the dam, that a major causeway would be constructed for the Highway #527 crossing, that a minor raising of the existing roadway would be adequate for the most northerly crossing and that the fourth crossing would be abandoned completely.

2.0 Site Investigation and Testing

This section presents the geological and geotechnical data gathered during the Level I investigation of the Pine Coulee Site. The investigation consisted of 67 borehole sites for a total of approximately 980 m of drilling. Interpretation of the data is discussed in the following sections of this report.


PROPOSED DAM
WILLOW
WILLOW CREEK
PROVINCIAL PARK
CREEK
PROPOSED PINE COULEE RESERVOIR
FORT McMURRAY
ATHABASCA
COLD LAKE
WESTLOCK
ST. PAUL
EDMONTON
LLOYDMINSTER
WETASKIWIN
CAMROSE
WAINWRIGHT
RED DEER
CORONATION
DRUMHELLER
CALGARY
BROOKS
PROJECT AREA
CLARESHOLM
MEDICINE HAT
LETHBRIDGE
CARDSTON
MOUNTAINS
SASKATCHEWAN
MONTANA

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
KEY PLAN

FIG 1

2.1 Summary of Investigations

Investigations began with an office study of available data which included geological and groundwater reports for the region, and previous site investigations near Willow Creek and Pine Coulee by the Prairie Farm Rehabilitation Administration (PFRA). This was followed by a preliminary airphoto and geological interpretation of the site. A borehole program was laid out by the Geotechnical Design Section and drilling commenced immediately after access permission was received.

2.1.1 Objectives

The objective of the investigation was to provide geological and geotechnical information for a Level I design of the Pine Coulee Site components. The main features to be identified were:

1) Overburden stratigraphy.
2) Surface profile of the bedrock.
3) Abutment and valley bottom material characteristics with regard to settlement, stability and seepage.

4) Preliminary pervious and impervious borrow sources.

2.1.2 Access and Approvals

The Pine Coulee Reservoir site is located in a farming community approximately 13 km southwest of Stavely. Access to the area is good along secondary Highway #527 and numerous other municipal roads.

Since all affected lands were privately owned, numerous initial delays were encountered in obtaining access agreements. All negotiations were conducted by the Land Services Section of the Geotechnical Branch. Land Use Agreements were signed by landowners and received by the Geotechnical Branch on March 25, 1985. Field investigations commenced on April 1, 1985.

Minimal clearing or dozer work was required thus helping to facilitate immediate commencement of drilling and limiting damage claims upon completion of the project.

2.1.3 Drilling

Drilling investigations were conducted by two Department drills (Mobile B50 & B61) at 67 borehole locations which are shown on Drawing Nos. 2, 3, and 4. The site investigation data for each of the boreholes is given in Table 1. This work began on April 1, 1985 and was completed by July 22, 1985. The investigations resulted in approximately 980 m of drilling. The borehole program commenced with auger drilling to determine the overburden stratigraphy. This was followed by core drilling around the damsite to prove and evaluate the bedrock.

A breakdown as to the type of drilling performed is as follows:

Table 1 PINE COULEE SITE INVESTIGATION DATA

Borehole	Elevation (m)	Location/Project Component	Total Depth (m)	Depth to Bedrock (m)	Drilling Type *	Sampling Type	Piezometer Zone (m)	Field Testing	Remarks
PC-1	1052.49	NW32-13-28-4 Weir	9.6	-----	HCA	Shelby/SPT	---------	----------	----------
PC-2	1051.66	NW32-13-28-4 Weir	8.0	8.0	HCA	Shelby/SPT	---------	----------	Auger Refusal at 8.0 m - suspected bedrock contact.
PC-3	1049.09	NW32-13-28-4 Weir	6.5	-----	HCA	Shelby/SPT	---------	----------	----------
PC-4	1049.30	NW32-13-28-4 Weir	6.4	-----	HCA	Shelby/SPT	0-5.35	Perm at 3.05 m in overburden	----------
PC-5	-------	NW32-13-28-4 Weir	---	-----	---	----------	---------	-------------	Cancelled
PC-6	1049.30	NW32-13-28-4 Weir	22.35	20.25	HCA	Shelby/SPT	---------	-------------	Bedrock-weathered clay shale. Flowing well condition at 18.3m approximately 80 gal/min.- hole plugged with inflatable bladder and cemented shut.
PC-7	1075.61	NW32-13-28-4 Weir	45.75	-----	HCA/SA	Shelby/SPT Bag	---------	-------------	Hole ended in sand and gravel. Suspected bedrock at approximately 46.0 m. Drilled two standard auger holes to 7.6 m for borrow material.
PC-8	1074.12	NE32-13-28-4 Weir	36.03	36.0	HCA	Shelby/SPT	30.0-38.4	-------------	Two holes drilled: - overburden sampling and bedrock contact. - piezometer installation.
PC-9	1052.12	NE32-13-28-4 Weir	14.3	12.2	HCA	Shelby/SPT	----------	-------------	Bedrock contact at 12.2 m. Auger refusal at 14.3 m.
PC-10	1049.30	NE32-13-28-4 Diversion Channel	3.3	1.6	HCA	Shelby/SPT	----------	-------------	Bedrock-sandstone.
PC-11	1056.90	NE32-13-28-4 Diversion Channel	8.0	-----	HCA	Shelby	----------	-------------	----------
PC-12	1053.17	NE32-13-28-4 Diversion Channel	3.0	-----	HCA	Shelby/Bag	----------	-------------	Auger refusal twice at 3.0 m -suspected bedrock contact.

* HCA - Hollow Core Auger

Borehole	Elevation (m)	Location/Project Component	Total Depth (m)	Depth to Bedrock (m)	Drilling Type	Sampling Type	Piezometer Zone (m)	Field Testing	Remarks
PC-13	1048.34	SW33-13-28-4 Diversion Channel	7.9	-----	HCA	Shelby	---------	----------	----------
PC-14	1059.86	NW33-13-28-4 Diversion Channel	15.35	-----	HCA	Shelby	---------	----------	----------
PC-15	-------	Diversion Channel	-----	-----	---	--------	---------	----------	Cancelled
PC-16	1055.47	SW33-13-28-4 Diversion Channel	7.95	-----	HCA	Shelby	---------	----------	-----------
PC-17	1057.85	SE33-13-28-4 Diversion Channel	9.55	-----	HCA	Shelby	---------	----------	-----------
PC-18	1067.97	SE33-13-28-4 Diversion Channel	15.55	-----	HCA	Shelby	---------	----------	-----------
PC-19	1061.27	SW34-13-28-4 Damsite (Borrow)	4.55	-----	SA	Bag	---------	----------	-----------
PC-20	1057.68	SW34-13-28-4 Damsite (Borrow)	4.55	-----	SA	Bag	---------	----------	------------
PC-21	1056.79	SW34-13-28-4 Inlet Chute	38.1	-----	HCA	Shelby/ SPT	24.0-30.5	----------	Hole abandoned as augers stuck in sand and gravel from 33.1 m to 38.1 m.
PC-22	1036.56	SW34-13-28-4 Inlet Chute	22.15	20.7	HCA	Shelby/ SPT	---------	----------	Bedrock-hard clay shale.
PC-23	1055.62	SW34-13-28-4 Dam Centreline	31.23	20.35	HCA/SA/ Rotary	Shelby/SPT/ Bag/Core	14.0-20.7	Perm at 3.05 m in overburden. Pressure test at 22.85-30.5 m in bedrock.	Bedrock-sandstone and clay shale. Three holes drilled: -sampling -perm test -peizometer installation.
PC-24	1035.05	SW34-13-28-4 Dam Centreline	14.65	12.2	HCA/ Rotary	Shelby/SPT/ Core	----------	-------------	Bedrock-sandstone
PC-25	1033.21	SW34-13-28-4 Dam Centreline	14.9	14.25	HCA	Shelby	----------	-------------	Bedrock-sandstone

Borehole	Elevation (m)	Location/Project Component	Total Depth (m)	Depth to Bedrock (m)	Drilling Type	Sampling Type	Piezometer Zone (m)	Field Testing	Remarks
PC-26	1033.29	SW34-13-28-4 Dam Centreline	15.0	14.1	HCA	Shelby	---------	----------	Bedrock-sandstone.
PC-27	1033.84	SW34-13-28-4 Dam Centreline	10.6	9.9	HCA	Shelby/SPT	---------	----------	Bedrock-sandstone
PC-28	1055.16	SW34-13-28-4 Dam Centreline	31.5	15.3	HCA/ Rotary	Shelby/SPT/ Core	---------	Perm at 1.5 m in overburden Pressure test at 15.25-31.5m in bedrock.	Bedrock-sandstone and clay shale. Three holes drilled: -sampling -perm test -pressure test
PC-29	1033.36	SW-34-13-28-4 Damsite	21.0	20.4	HCA	Shelby/SPT	---------	-----------	Bedrock-sandstone. Flowing well condition at 6.1 m - approximately 2 gal/ min - hole cemented.
PC-30	1033.06	SW34-13-28-4 Damsite	10.0	9.25	HCA	Shelby/SPT	---------	-----------	Bedrock-sandstone.
PC-31	1049.51	NE27-13-28-4 East Access Road	6.9	6.3	HCA	Shelby	---------	-----------	Bedrock-silty shale. Auger refusal at 6.9 m.
PC-32	1049.06	SE34-13-28-4 Saddle Dam	7.8	----	HCA	Shelby	---------	-----------	----------
PC-33	1043.10	NW35-13-28-4 Saddle Dam	15.55	----	HCA	Shelby	---------	-----------	----------
PC-34	1071.33	NW32-13-28-4 Weir	32.6	30.82	HCA/ Rotary	Shelby/SPT/ Core	26.0-32.6	-----------	Bedrock-sandstone. Two holes drilled: -sampling and bedrock contact -perm test
PC-35	1051.06	NE32-13-28-4 Diversion Channel	9.75	9.75	HCA	Shelby/SPT	----------	-----------	Bedrock-sandstone.
PC-36	1052.56	SW33-13-28-4 Diversion Channel	8.0	----	HCA	Shelby	----------	-----------	----------

Borehole	Elevation (m)	Location/Project Component	Total Depth (m)	Depth to Bedrock (m)	Drilling Type	Sampling Type	Piezometer Zone (m)	Field Testing	Remarks
PC-37	1044.67	SW33-13-28-4 Diversion Channel	8.0	----	HCA	Shelby	---------	----------	----------
PC-38	1057.25	SE33-13-28-4 Diversion Channel	9.55	----	HCA	Shelby	---------	----------	----------
PC-39	1061.25	SE33-13-28-4 Diversion Channel	15.7	----	HCA	Shelby	---------	----------	----------
PC-40	1057.36	SW34-13-28-4 Inlet Chute	45.0	43.3	HCA/ Rotary	Shelby/SPT	---------	----------	Bedrock-sandstone. Two holes drilled: -sampling -bedrock contact established with rotary drilling from 39.65 to 45.
PC-41	1039.13	SW34-13-28-4 Inlet Chute	24.5	23.7	HCA	Shelby/SPT	---------	----------	Bedrock-clay shale.
PC-42	1033.69	SW-34-13-28-4 Damsite	15.4	15.0	HCA	Shelby/SPT	---------	Vane Shear at 2.6-11.75m	Bedrock-clay shale. Two holes drilled: -sampling -vane shear test Flowing well condition at 15.0 m - approximately 15 gal/min, hole cemented.
PC-43	1033.68	SW34-13-28-4 Damsite	19.9	19.0	HCA	Shelby/SPT	---------	Vane Shear at 4.1-11.75m	Bedrock-clay shale. Two holes drilled: -sampling -vane shear test
PC-44	1045.59	SE-15-14-28-4 Highway 527 Crossing	7.65	1.0	HCA	Shelby/SPT	---------	----------	Bedrock-sandstone and clay shale.
PC-45	1039.02	SE15-14-28-4 Highway 527 Crossing	11.95	10.0	HCA	Shelby/SPT	---------	----------	Bedrock-sandstone and clay shale.
PC-46	1043.42	SE-15-14-28-4 Highway 527 Crossing	16.8	11.6	HCA	Shelby/SPT	---------	----------	Bedrock-clay shale

Borehole	Elevation (m)	Location/Project Component	Total Depth (m)	Depth to Bedrock (m)	Drilling Type	Sampling Type	Piezometer Zone (m)	Field Testing	Remarks
PC-47	1063.88	SE15-14-28-4 Highway 527 Crossing	4.55	1.3	HCA	Shelby/SPT	---------	----------	Bedrock-clay shale.
PC-48	1038.68	NE10-14-28-4 Highway 527 Crossing	10.95	9.25	HCA	Shelby	---------	----------	Bedrock-clay shale.
PC-49	1073.88	SE-15-14-28-4 Highway 527 Crossing (Borrow)	3.1	3.1	SA	Bag	---------	----------	Auger refusal at 3.1 m -suspected bedrock contact
PC-50	1076.39	SW15-14-28-4 Highway 527 Crossing (Borrow)	5.0	----	SA	Bag	---------	----------	----------
PC-51	1076.96	SW15-14-28-4 Highway 527 Crossing (Borrow)	5.0	----	SA	---------	---------	----------	----------
PC-52	1075.21	SW15-14-28-4 Highway 527 Crossing (Borrow)	5.0	----	SA	Bag	---------	----------	----------
PC-53	1078.14	NE10-14-28-4 Highway 527 Crossing (Borrow)	5.0	----	SA	Bag	---------	----------	----------
PC-54	1033.21	SW34-13-28-4 PFRA Dam Centreline	17.06	10.1	HCA/ Rotary	Shelby/SPT/ Core	---------	----------	Bedrock-sandstone and clay shale.
PC-55	1033.18	SW34-13-28-4 PFRA Dam Centreline	20.06	14.8	HCA/ Rotary	Shelby/SPT/ Core	---------	----------	Bedrock-sandstone.
PC-56	1048.10	NW35-13-28-4 Saddle Dam	15.65	----	HCA	Shelby/SPT	---------	----------	----------
PC-57	1048.61	NW35-13-28-4 Saddle Dam	12.25	12.0	HCA	Shelby/SPT	---------	----------	Bedrock-shale. Auger refusal at 12.25 m.

Borehole	Elevation (m)	Location/Project Component	Total Depth (m)	Depth to Bedrock (m)	Drilling Type	Sampling Type	Piezometer Zone (m)	Field Testing	Remarks
PCB-1	1069.86	NW34-13-28-4 Damsite (Borrow)	5.35	-----	SA	Bag	---------	----------	----------
PCB-2	1060.66	NW34-13-28-4 Damsite (Borrow)	5.35	-----	SA	Bag	---------	----------	----------
PCB-3	1060.61	NW34-13-28-4 Damsite (Borrow)	5.35	-----	SA	Bag	---------	----------	----------
PCB-4	1057.98	NW34-13-28-4 Damsite (Borrow)	5.35	-----	SA	Bag	---------	----------	----------
PCB-5	1113.24	SW4-14-28-4 Damsite (Borrow)	5.4	3.7	SA	Bag	---------	----------	Bedrock-clay shale. Auger refusal at 5.4 m.
PCB-6	1119.34	SW4-14-28-4 Damsite (Borrow)	4.25	3.7	SA	Bag	---------	----------	Bedrock-clay shale. Auger refusal at 4.25 m.
PCB-7	1108.29	SW4-14-28-4 Damsite (Borrow)	5.8	5.8	SA	Bag	---------	----------	Auger refusal at 5.8 m.
PCB-8	1098.12	SW4-14-28-4 Damsite (Borrow)	3.0	3.0	SA	Bag	---------	----------	Auger refusal at 3.0 m.
PCB-9	1093.54	E½6-14-28-4 Damsite (Borrow)	6.0	-----	SA	Bag	---------	----------	----------
PCB-10	1108.19	E½6-14-28-4 Damsite (Borrow)	6.0	-----	SA	Bag	---------	----------	----------
PCB-11	1105.21	E½6-14-28-4 Damsite (Borrow)	6.0	-----	SA	Bag	---------	----------	----------
PCB-12	1114.50	E½6-14-28-4 Damsite (Borrow)	6.0	-----	SA	Bag	---------	----------	----------

1) 793.65 m overburden drilling with hollow core auger including vane shear, permeability tests and piezometer installations.

2) 37.77 m core drilling with a N.W. barrel and diamond bit.

3) 103.65 m borrow investigation with standard flight auger.

4) 43.15 m rockbit or hollow core auger to provide casing for core holes.

2.1.4 Sampling

Various types of samples were retrieved depending on the type of hole drilled and information required. Shelby tube (75 mm I.D.) and standard penetration test (51 mm O.D.) samples were taken in the hollow core auger holes and bag samples in the borrow holes drilled with a solid stem auger. Continuous cores were recovered in the rotary holes. All samples were field classified and preserved for lab logging and testing.

2.1.5 Field Testing

Field vane shear tests were carried out in boreholes PC-42 and 43 in the clay layers ranging from 2.6 m to 11.75 m below the Pine Coulee floodplain. The equipment used was an "Acker Precision Torque Head" field vane shear unit.

Permeability testing was conducted in boreholes PC-4, 23, 28 and 34. Falling head testing was performed in one overburden zone for each of the above holes. Pressure permeability testing was carried out in the bedrock in holes PC 23 and 28. The pressure testing unit used was a double inflatable packer system which isolates a 1.5 m zone of borehole. The pressure testing consisted of a total of 15 tests in the two boreholes.

To identify pore pressures and groundwater levels, piezometers were installed in five boreholes, PC-4, 8, 21, 23 and 34. All piezometers were 25.4 mm PVC standpipes with slotted tips. Flowing well (artesian) conditions were encountered in layers of gravelly sand near the bedrock within boreholes PC-6, 29 and 42. Due to the difficulties involved in installing piezometers through a hollow core auger with artesian flow conditions, no piezometers were installed in these holes at this time. If piezometers are required in the future, a procedure will have to be devised to control flow and successfully complete the installations.

2.1.6 Borrow Investigations

An impervious borrow investigation was carried in three areas with a total of 12 boreholes drilled. These boreholes were designated as PCB 1 to 12. In addition, a pervious borrow search was initially conducted within a 50 km radius of the site and later expanded to 100 km. This search started with a review of the Geotechnical Services Section granular files followed by a review of the Alberta Transportation Aggregate Section files. Two field trips were subsequently made to check and sample prospective sources. The granular search was completed by October 1985.

2.1.7 Site Reclamation

All boreholes were backfilled or grouted upon completion of the field investigation. Site reclamation was carried out to the satisfaction of the landowners. Any damages that were unable to be repaired at that time were compensated for through negotiations between the landowner and the Land Services Section of the Geotechnical Branch.

2.2 Laboratory Testing

The samples of soil and bedrock retrieved from the boreholes were delivered to the Alberta Environment, Geotechnical Services Section, Materials Testing Laboratory in Edmonton. Here, the samples were extruded, visually inspected, and classified. As well, specimens were selected for design testing to provide the geotechnical parameters necessary for a feasibility grade design.

2.2.1 Laboratory Description and Classification

All samples including the core were carefully examined and described from visual observations. For each auger hole sample and run of bedrock core, the percent recovery was estimated and where applicable tested with a pocket penetrometer (Soil test CL 1700). The natural moisture content of every sample and the wet unit weight of undisturbed cohesive samples was determined.

Soil classifications were given to the samples based on particle size analysis and Atterberg Limits in accordance with the Modified Unified Soil Classification System. Particle size analysis were carried out when possible at every suspected change of material or generally at 3 m intervals in uniform deposits. In plastic soils, the Atterberg Limits were determined in conjunction with the particle size analysis.

The laboratory descriptions, classifications and associated test results were recorded on detailed borehole logs. The borehole logs are available within the Geotechnical Services Sections.

2.2.2 Design Testing

Design testing was performed on selected specimens of soil and bedrock to provide specific geotechnical parameters for design considerations. Copies of the test results are available within Geotechnical Services Sections.

Settlement parameters were derived by performing 12 oedometer consolidation tests. These were done on 10 specimens of the valley bottom clays obtained from beneath the proposed Pine Coulee damsite as well as on two specimens of the valley bottom clays taken in the vicinity of the Highway #527 crossing.

In order to establish the shear strength parameters of the damsite foundation materials, direct shear tests were performed on six specimens of the valley bottom clays. In addition, undrained shear strengths were determined through unconfined compression tests performed on 10 specimens of valley bottom clays taken from various boreholes upstream and downstream of the proposed dam centreline as well as on one specimen of the foundation clay taken in the vicinity of the Highway #527 Crossing.

Lastly, a series of tests were performed to investigate the suitability of the locally available till deposits as potential impervious borrow materials. Two consolidated undrained triaxial tests with pore pressure measurements were performed on combined remoulded clay till samples taken from the proposed borrow area on the west side of Pine Coulee upstream of the damsite. These samples were compacted at optimum moisture content (O.M.C.) and at plus 2% of the O.M.C. The remainder of the testing involved unconfined compression tests on nine proctors of clay till taken from various boreholes near the abutments of the damsite and Highway #527 crossing.

3.0 General Geology

The geological conditions described in the following Sections are based on published literature and maps as well as interpretation of the information obtained from the Pine Coulee Site investigations.

3.1 Bedrock Geology

The bedrock geology of southwestern Alberta has been mapped by Irish (1971) and Green (1972). As shown by Irish (1971), the Willow Creek-Pine Coulee area is underlain by strata of the Porcupine Hills Formation which is Lower Tertiary in geological age. These strata primarily consist of interbedded grey sandstone and grey mudstone or shale. They are underlain by the Lower Tertiary/Upper Cretaceous Willow Creek Formation which gently rises to form the bedrock surface under the overburden approximately 3 km to 5 km east of the project site. Strata of the Willow Creek Formation consist of grey sandstone interbedded with grey, green or pink, clayey to bentonitic mudstone or shale with a few thin beds of limestone.

Both the Porcupine Hills and Willow Creek Formations lie on the eastern limb of the Alberta Syncline which has its axis in the foothills to the west of the project site. From this axis, the bedrock surface to the east has a gentle west to southwest dip of 1 m to 2 m/km. On a local basis such as at the project site, the strata can be considered as essentially flat lying.

3.2 Bedrock Topography

In southwestern Alberta the bedrock surface topography has been interpreted by Geiger (1965, 1968). In the Willow Creek-Pine Coulee area, the configuration of the bedrock surface can be divided into the foothills belt and the lowland valley or plains area. The eastern edge of the foothills belt begins 8 km to 11 km west of the project site where the bedrock rises to form the irregular hills and ridges of the Porcupine Hills. The lowland valley or plains area lies to the east of the foothills and is characterized by broad valleys in the bedrock surface separated by erosional remnants comprising bedrock high areas. As interpreted by Geiger (1968) these are preglacial river valleys which generally trend east to northeast. They are infilled with preglacial and glacial sediments which masks most of the surficial expression of their existence. In particular, the preglacial Stavely Valley is interpreted as running under Pine Coulee from west to east through the N 1/2 34-13-28-W4, approximately 1 km north of the damsite.

The bedrock surface is also characterized by south to southeast trending valleys which are partly infilled with late glacial fluvial sediments. These valleys were probably formed by ice-marginal meltwater channels whose flow direction was controlled by the position of the retreating stagnant ice mass. Surficial expressions of these valleys are common in the Willow

Creek-Pine Coulee area with small creeks occupying wide U-shaped valleys (Drawing No. 1).

3.3 Surficial Geology

Surficial deposits include preglacial, glacial, late glacial and post glacial sediments. The most complex sequence lies over the buried preglacial valleys where surficial deposits can range up to 60 m in thickness. Over bedrock high areas however, this thickness can decrease to less than 1 m. The deposits which form the ground surface in the Willow Creek-Pine Coulee area are mapped on Drawing No. 1 and stratigraphic sections of the area are shown on Drawing Nos. 6 to 15.

3.3.1 Preglacial Sediments

The preglacial sediments are confined to the bottom of the buried valleys. These sediments are fluvial in origin. The sequence typically begins with interbedded sand and gravel resting on bedrock. These granular layers are usually overlain by clays interlayered with thin beds of silt or clayey sand which underlies glacial till.

As reported by Geiger (1968), the thickness of the basal sand and gravel layer can be more than 15 m in the Willow Creek-Pine Coulee area. In boreholes on the west side of Pine Coulee, up to 10 m of sand and gravel was encountered overlying bedrock (Drawing Nos. 8

to 10). The sand is medium to dark grey, clayey, silty and sometimes gravelly, dense to very dense and usually poorly graded. The gravels are olive to medium grey, clayey to silty with angular to subrounded rock fragments up to 40 mm diameter.

The overlying preglacial clays range up to 11 m in thickness. These clays are primarily of low to medium plasticity. They are intermittently interbedded with the layers of clayey silt, or clayey to silty sand usually less than 3 m in thickness. The clays are olive to dark grey, stiff to hard and often slightly overconsolidated. They also range from being massive to laminated and contain slickensides usually within pockets or lenses of high plastic clay.

3.3.2 Glacial Sediments

Glacially derived till overlies the preglacial sediments in the buried valleys and bedrock throughout most of the upland area east of the Porcupine Hills (Drawing No. 1). The thickest of these till deposits lie over the buried preglacial valleys. On the west side of Pine Coulee, the overlying glacial till attains a thickness of 35 m and extends to the upland ground surface (Drawing No. 10). Elsewhere the thickness decreases to less than 5 m and in places bedrock outcrops, particularly to the west of Pine Coulee and Willow Creek. The till is predominantly a low plastic clay interlayered with lenses of silt generally up to 3 m thick and a few sand and/or gravel lenses less than 2 m thick. The clay till is medium brown to light grey near the ground surface but changes to medium grey with depth. It is very stiff to hard, usually fissured and contains a variety of rock fragments ranging from sand to boulder size.

3.3.3 Late Glacial Sediments

Late glacial deposits include water modified or eroded till, outwash and glaciofluvial sediments.

The water modified till forms part of the upland surface to the west of Pine Coulee and Willow Creek (Drawing No. 1). It consists of a shallow deposit of low to medium plastic clay interbedded with layers of silt, sand and gravel (Drawing No. 10). The surficial characteristics include a water streamlined ground surface with scattered boulders in areas where the till surface was extensively eroded.

The outwash sediments are glaciofluvial in origin but their deposition was not topographically controlled. They occur scattered across the uplands and are most common near meltwater channels (Drawing No. 1). A large deposit of these sediments extends from NW 32-13-28-W4 to SW 5-14-28-W4, east and north of the Willow Creek weir site. They primarily consist of interbedded sand and gravel. As indicated by borehole PC-8 drilled into this deposit, the sand and gravel extends to a depth of 12 m and is medium brown, silty to clayey and contains subrounded rock fragments up to 40 mm in diameter. This deposit also likely contains some boulder size rock fragments.

Glaciofluvial sediments were also deposited by glacial meltwaters but their deposition was topographically controlled as they primarily exist within meltwater channels. In the Willow Creek-Pine Coulee area, these sediments partly infill Pine Coulee, an abandoned meltwater channel 3 km to 4.5 km to the west of Pine Coulee, and Willow Creek downstream of its confluence with the abandoned meltwater channel in N 1/2 28-13-24-W4 (Drawing No. 1). Within Pine Coulee and the abandoned meltwater channel, as much as 10 m of these sediments overlie bedrock and extend up to the present floodplain surface. These sediments primarily consist of low to high plasticity clay interbedded with layers of clayey to silty sand and a few layers of silt. The clays are light to medium grey and sometimes mottled brown and grey near the floodplain surface. They are usually soft to very soft and normally consolidated except near the floodplain surface where the upper layers are firm to hard and sometimes slightly overconsolidated. These upper layers have probably been dessicated due to the arid climate in this region. The layers of sand and silt interbedded with the clays range up to 5 m in thickness although they are more commonly 1 m to 2 m thick. The sand is primarily medium grey, silty, medium dense to dense and contains the odd pebble.

Under the Willow Creek floodplain downstream of the abandoned meltwater channel, glaciofluvial sediments extend to depths of 10 m to 15 m where they overlie bedrock. These sediments primarily consist of a clayey to sandy gravel.

3.3.4 Postglacial Sediments

Postglacial alluvial sediments form the floodplain surface along the major creek and river valleys while colluvium covers most of the valley slopes as well as the slopes of the Porcupine Hills (Drawing No. 1).

The terraces within the Willow Creek valley upstream of the abandoned meltwater channel and the floodplain all along the valley consist of alluvial sediments. Boreholes in a terrace on the south side of the Willow Creek weir site encountered alluvial sediments to a depth of 5 m to 6 m. These sediments overlie a thin layer of till probably less than 5 m thick which in turn overlies bedrock (Drawing No. 13). The alluvial sequence within this terrace consists of 1 m to 3 m of interbedded clay and silt overlying up to 5 m of sand and gravel. The clays and silts are medium to dark brown, hard and contain the odd pebble. The clays are low plastic and generally slightly overconsolidated probably due to dessication. The underlying sand and gravel is medium brown to medium grey, silty and dense to very dense. Rock fragments up to 80 mm in diameter were encountered. Beneath the sand and gravel, the upper layer of till appears to be intermixed with lenses and/or pockets of alluvial clay and silt probably a result of water erosion or reworking during the formation of the terrace.

In the Willow Creek floodplain downstream of Pine Coulee, the depth of alluvium is uncertain as these sediments become gravelly and similar to the underlying glaciofluvial sediments.

Colluvium has developed along most of the valley slopes within the area except for those in which oversteepening has occurred due to toe erosion. At the base of the slopes where the colluvium has collected, the thickness probably ranges up to 1.5 m. the colluvium generally consists of intermixed, weathered soil and bedrock resulting in a sandy to silty clay material which is mottled brown-grey, containing some organics, the odd pebble and a reworked or blocky texture.

3.4 Geomorphic Development

As interpreted by Geiger (1968), weathering and erosion in the Tertiary period prior to the Quarternary glaciations produced a mature landscape within the bedrock surface. This landscaping was similar to its present configuration with creeks and rivers draining eastward from the foothills out into the plains where they carved broad valleys into the bedrock. These rivers deposited alluvial sand and gravel along the floodplain of their bedrock valleys.

The advance of Quarternary ice sheets into the area blocked the easterly flowing rivers creating proglacial lakes in the broad river valleys. As a result lacustrine clays, silts and clayey sands were deposited over the alluvial sediments. These lacustrine deposits were subsequently overridden and partly eroded by the advancing ice sheets. The resulting glacial loading and unloading probably produced the noted slickensided zones.

Glacial events in the Quarternary period have been interpreted by Harris and Waters (1977). The sequence of events and associated deposits are listed in Table 2. It can be seen that southwestern Alberta was subjected to at least three glaciations each separated by an interglacial period. Each glacial event was marked by an advance of a Cordilleran or Rocky Mountain ice sheet from the west. This was followed by an advance from the east or northeast of a Laurentide or Continental ice sheet. These two ice sheets generally meet along a north-south line passing through the Willow Creek-Pine Coulee area. In each event, the Cordilleran ice sheet barely reached the Willow Creek-Pine Coulee confluence and was

Table 2 <ins>SIMPLIFIED SEQUENCE OF GLACIAL EVENTS IN SOUTHWESTERN ALBERTA</ins>

(after Harris and Waters 1977)

Event	Possible Age	Glaciation	Source	Deposits
Glacial Event III	Illinoian and earlier more than 145,000 years B.P.	Albertan	Cordilleran	Albertan Till
		Labuma	Laurentide	Labuma Till
Non Glacial Event III-II	Sangamon		Fluvial Deposition	Sand and Gravel
Glacial Event II	Illinoian or early Wisconsin	Maycroft	Cordilleran	Maycroft Till
		Maunsell	Laurentide	Maunsell Till
Non Glacial Event II-I	Port Talbot 18,000-33,000 year B.P.		Fluvial Deposition	Silts and Sands
Glacial Event I	Late Wisconsin more than 7,000 to 9,600 years B.P.	Bow Valley	Cordilleran	Bow Valley Tills
		Lochend	Laurentide	Lochend Till

already in retreat as the later Laurentide ice sheet reached its maximum extent to the west of Willow Creek and Pine Coulee. These sequential advances resulted in a zone of overlapping glacial sediments with deposition of at least one Cordilleran till and one Laurentide till during each of the three glaciations. As a result, there is the potential for six till sheets to exist in the Willow Creek-Pine Coulee area.

Ideally the tills deposited during each glaciation should be separated by interglacial alluvial or lacustrine sediments consisting of fluvial sands, silts or clays. As well, the till surfaces being exposed to subaerial weathering and erosion should have developed a dessicated surface or paleosol. However, due to subsequent glacial advances, these interglacial sediments as well as the previously deposited tills could have been partly or completely eroded by the overriding ice sheet. Possibly some of the sand and silt lenses encountered within the tills at the project site could represent remnants of these interglacial sediments, although detailed stratigraphic and lithologic work would be required to distinguish these sediments as well as different till sheets.

Late glacial and postglacial processes have molded the present natural ground surface to its existing shape. Evidence suggests that the last glaciation ended some 9000 years ago with the retreat and stagnant disintegration of the Laurentide ice sheet (Harris and Waters, 1977). The resulting ablation process produced large quantities of meltwater which flowed down over the freshly deposited till modifying and in some places completely eroding it leaving lag boulders scattered on the ground surface. As well, outwash sediments of sand and gravel became deposited over the till. The continued flow of the meltwaters eventually began downcutting channels leaving several terrace levels. The direction at which these channels became orientated was controlled by the position of the ablating ice sheet which blocked an easterly drainage forcing the meltwaters to flow southward along the ice front. Most of these channels were downcut through the till and into the underlying bedrock. It is conceivable that during the downcutting process slumping may have occurred on slopes of the meltwater channels. At the maximum point of downcutting, these slopes were 10 m to 15 m higher and probably steeper than present. As well, the groundwater table was probably at a higher elevation in the slopes. These conditions would aggravate instability resulting in slump blocks being intermixed with the subsequently deposited glaciofluvial sediments and eventually buried under the floodplain.

Due to changes in the flow regime, at some point downcutting of these channels ceased and they began to infill with glaciofluvial sediments. This was in turn followed by postglacial deposition of alluvium in terraces and floodplains along the major creek valleys. These sediments are still being deposited in lateral and point bars as the creek meanders across the valley as well as on the floodplain during seasonal flood stages.

During late glacial and postglacial times, colluvium developed along creek valley slopes as well as on the slopes of the Porcupine Hills resulting from gravity induced movements of weathered soil and/or bedrock material from the inclined surfaces of the slopes. In the Willow Creek-Pine Coulee area, the primary type of downslope movement has been creep although there has been some slumping along the Willow Creek valley slopes due to undercutting and oversteepening along the outsides of meanders.

3.5 Hydrogeology

The regional hydrogeological conditions of the Willow Creek-Pine Coulee area are discussed in an Alberta Research Council report by Ozoray and Lytviak (1974). In addition, a site specific report providing a detailed hydrogeological assessment of the project site has been prepared by Piteau Engineering Ltd. (1985).

As described by Piteau Engineering Ltd. (1985), the groundwater surface elevations generally follow the topography of the bedrock surface. Where there is a bedrock high, the groundwater surface elevation is correspondingly high. The groundwater surface elevation approximately 1.2 km west of Pine Coulee is around 1085 m while 1.5 km to the east the groundwater surface is somewhat lower at an elevation of around 1048 m. It should be noted however, that Piteau Engineering Ltd. (1985) has indicated that these elevations are likely lower than normal primarily due to an extended dry period prior to and during the time of measurement. Horizontal groundwater movement is generally towards Pine Coulee while within the valley bottom, the general movement is downward which indicates that Pine Coulee acts as a recharge area.

Permeability testing performed on the various geological formations by Piteau Engineering Ltd. (1985) indicate that the clay tills have a low hydraulic conductivity ranging from approximately 10^{-7} to 10^{-10} m/sec. The lenses of sand and/or silt which are present within the till have a higher hydraulic conductivity in the order of 10^{-5} to 10^{-6} m/sec. The glaciofluvial silty clays within the bottom of Pine Coulee also have a low hydraulic conductivity ranging from approximately 10^{-6} to 10^{-9} m/sec, while the underlying bedrock strata of interbedded sandstone and shale have a conductivity of 10^{-8} to 10^{-9} m/sec. The preglacial Stavely Valley which crosses beneath Pine Coulee in the N 1/2 34-13-28-W4 appears to drain eastward. Test results indicate that the basal sand and gravel buried within this valley creates a confined aquifier with an excess head of from 7.5 m to 12 m and a hydraulic conductivity of 10^{-6} to

10^{-7} m/sec. The conductivity of this aquifier may however be somewhat higher elsewhere along the buried channel as the tests were limited to only a small segment of the channel.

4.0 Engineering Geology

4.1 Diversion Weir Site

The stratigraphy of the diversion weir site on Willow Creek is shown on Drawing Nos. 13 and 14. At this location, Willow Creek flows around a broad meander and is undercutting the left abutment slope. This abutment slope is 25 m to 30 m high, has an inclination of between 1.5H:IV and 2H:IV and consists primarily of a low plastic clay till. The ongoing undercutting is causing shallow slumping as the slope is repeatedly oversteepened.

The inside of the meander is occupied by an alluvial terrace across which the diversion dyke is to be constructed. This terrace extends approximately 450 m across the valley bottom from the edge of the creek to the bottom of the south valley slope. The terrace surface lies at 2 m to 3 m above the creek level and gently slopes upward away from the creek. The surface of the terrace consists of a hard dessicated layer of interbedded low plastic clay and silt ranging in thickness from 1 m to 3 m. This dessicated layer is underlain by an interbedded sand and gravel layer of less than 5 m in thickness which in turn overlies a clay till. The sands and gravels are dense to very dense and rock fragments ranging up to 80 mm in diameter were encountered in borehole PC-3. The silt and clay content of the sand and gravel varies from 5% to 18% with an average of approximately 10% which results in a relatively permeable foundation for the diversion dyke and weir structure.

At the actual proposed weir site, the Willow Creek channel is 10 m to 15 m wide and appears to be cut into hard impervious clay till. Borehole PC-6 which was approximately 190 m downstream of the proposed weir in the valley bottom indicated this clay till is 5 m in thickness and that it is underlain by a sequence of interbedded silty to gravelly sands and low plastic, hard clay. Artesian water was encountered at a depth of 18 m where this borehole penetrated the lower layer of sand. Below the left abutment slope approximately 100 m downstream of the proposed weir site, borehole PC-7 encountered a similar sequence of gravelly sands, clays and silts underlying till at a depth of 36 m. As shown on Drawing No. 14, a correlation between the sediments within PC-6 and PC-7 can be inferred. This evidence indicates that these underlying sediments which are probably preglacial fluvial deposits, may extend all the way upstream to the proposed weir site.

4.2 Diversion Channel

Stratigraphic cross sections for the upstream end of the diversion channel are shown on Drawing Nos. 13 and 14. From the left abutment of the weir site, the proposed diversion channel will be cut through a narrow ridge 25 m to 35 m high and 25 m to 100 m across its crest. This ridge separates the Willow Creek valley from an adjacent abandoned meltwater channel. It generally consists of a low plastic clay till although the upland surface is partly formed by deposits of outwash sediments consisting primarily of sand and gravel. As indicated by boreholes PC-8 and PC-34, these outwash sediments extend to a depth of 7 m to 12 m and may contain perched water tables. No major excavation or stability problems are anticipated for the 26 m deep cut required for the diversion channel crossing of this ridge.

On the east side of the ridge, the abandoned meltwater channel is some 600 m to 800 m wide and 12 m to 23 m deep along the proposed diversion channel crossing. Boreholes in the valley bottom indicate that this channel has been partly infilled with 10 m to 2 m of glaciofluvial sediments overlying bedrock. These sediments consist primarily of relatively impervious clays and silts although there are some local pockets of more permeable sand and gravel as shown by borehole PC-12. The clays near the floodplain surface are hard and slightly overconsolidated probably as a result of dessication.

In the central portion of the valley, a water-streamlined terrace exists with intermittent creeks flowing down each side. This terrace is from 150 m to 200 m across its crest and rises 5 m to 7.5 m above the valley bottom with gently inclined side slopes of from 6.5H:IV to 12H:IV. Where the diversion channel crosses the terrace, there appears to be up to 5 m of glaciofluvial clays and silts overlying a low to medium plastic clay till. The till is expected to be less than 10 m thick and overlying bedrock.

In the vicinity of the diversion channel crossing, there is no obvious evidence of slope movement along the valley banks on either side of the abandoned meltwater channel.

To the east of the meltwater channel, the diversion channel crosses some 1500 m of upland before reaching the proposed inlet chute to the Pine Coulee reservoir. This upland area generally consists of low plastic clay till. The excavation for the diversion channel may however encounter lenses to silt, sand and/or gravel with perched water tables within the till. This water will drain once exposed by the excavation as they are probably not extensive and are surrounded by impervious fill.

On the west abutment of Pine Coulee just prior to the inlet chute, the diversion channel will be excavated through northeast to southwest trending drumlinoid ridges (Drawing Nos. 1 and 2).

These ridges are up to 15 m high and extend from SE 33 to NW 34-13-28-W4. Boreholes PC-39 and PCB-1 which were drilled into these ridges encountered up to 5.5 m of sand and gravel overlying till. The proposed invert of the diversion channel excavation will extend well into this underlying impervious till and therefore, extensive seepage losses are not anticipated.

4.3 Damsite

The stratigraphy across Pine Coulee and along the valley slopes in the vicinity of the damsite is shown on Drawing Nos. 6 to 12. In addition, the topography of the bedrock surface interpreted from boreholes drilled with the damsite area is shown on Drawing No. 5.

Pine Coulee at the proposed damsite has a U-shaped configuration. The valley bottom is flat and approximately 275 m to 300 m across while the valley banks are approximately 20 m in height. The underlying bedrock surface across the damsite is also U-shaped. Within the abutments, this bedrock surface lies between 2 m to 7 m above the floodplain level while beneath the valley bottom, it drops to a depth of approximately 14 m below floodplain level. The bedrock is overlain by up to 20 m of till in the abutments while in the valley bottom, glaciofluvial sediments infill the U-shaped bedrock channel to the present floodplain level.

4.3.1 Abutment Conditions

The left abutment slope rises at approximately 4H:IV for 21 m above the Pine Coulee floodplain level. The overburden-bedrock contact exists at approximately 6 m above floodplain level with approximately 15 m of overlying till extending to the upland ground surface. The right abutment slope rises at approximately 5H:IV for a height of 20 m above the floodplain. Here the overburden-bedrock contact is less than 3 m above the floodplain level with approximately 20 m of overlying till extending to the upland ground surface.

The bedrock within both abutments consists of interbedded sandstone and shale layers probably of the Porcupine Hills Formation. The upper portion of the bedrock to a depth of approximately 7 m below the overburden contact primarily consists of a clayey to silty shale. This shale is weathered and rust stained, moderately fractured to intact and contains intermittent slickensided zones. Sandstone underlies the shale. It is generally fine to medium grained and moderately fractured to intact with a few badly fractured zones.

The low plastic clay till which overlies the bedrock within both abutments is hard, fissured and contains several lenses of silt and sand particularly within the right abutment. These lenses are not anticipated to be significant from a seepage control point of view since they are discontinous in nature and generally have a low hydraulic conductivity ranging from 10^{-5} to 10^{-8} m/sec. The average silt and clay content of the lenses is approximately 30%

Both of the abutment slopes are covered with a sandy to silty clay colluvium varying from 1.5 m thick near the valley bottom to somewhat less near the valley crest. These colluvial materials consist of loose reworked sandy to silty clay with some organics and pebbles. It would appear that they may require excavation within the vicinity of the proposed dam.

The slickenside zones within the shale and the fissures within the overlying till would appear to be primarily due to valley rebound. This rebound could be caused by loading and unloading and/or by downcutting of the Pine Coulee meltwater channel. There is no evidence either surficial or within the foundation stratigraphy indicating that slope movements at the damsite could have developed the slickensides and fissures.

4.3.2 Foundation Conditions

Beneath the proposed dam centreline, as much as 14 m of glaciofluvial sediments infill a U-shaped bedrock channel. The bedrock channel appears to have a thalweg with a bottom elevation of 1018 m which runs along the east side of the existing valley

(Drawing No. 5). This thalweg was probably formed by the late glacial meltwaters which downcut Pine Coulee. Some 100 m to 200 m upstream of the proposed dam centreline, the bedrock surface slopes downward some 5 m to 6 m to an approximate elevation of 1013 m. This sloping bedrock may be evidence of the southern side of the preglacial Stavely Valley which runs west to east. The foundation materials which underlie the Pine Coulee glaciofluvial sediments in this bedrock low consist of up to 10 m of interbedded sand and gravel overlain by 11 m of clay containing thin beds of silt or clayey sand. Significant features of these preglacial sediments is that the layer of sand and gravel has an excess hydraulic head of 7 m to 12 m and in some places artesian water pressures. As well, the overlying clays are slightly overconsolidated and contain several zones of slickensides.

The bedrock beneath the proposed dam centreline consists of hard, fine to medium grained sandstone. The sandstone is weathered and contains the odd thin bed of weathered shale. The depth of weathering was not confirmed although it probably extends deeper than the 3 to 4 m that were cored. This weathering probably occurred due to the prolonged exposure of the bedrock surface to subaerial erosion during preglacial times. It is possible that this sandstone strata may belong to the Willow Creek Formation which underlies the Porcupine Hills Formation.

The overlying sequence of glaciofluvial sediments generally consists of medium plastic clays interbedded with layers or lenses of silt and sometimes sand. The clays vary from soft to very stiff and are generally normally consolidated except for a 2 m to 3 m thick layer which exists near the floodplain level. This layer is firm to hard and appears to be slightly overconsolidated as a result of dessication. At a depth of approximately 5 m, a weak foundation layer consisting of soft to firm clay of low to intermediate plasticity can be delineated. This layer varies from 1 m to 4 m in thickness. In addition, below 7 m in depth, the clays contain the odd, random slickenside possibly the result of freeze-thaw cycling during deposition. The sand and silt layers interbedded with the clays are generally less than 2 m thick and appear to be discontinuous. The silt and clay content of these layers is generally greater than 25% which also indicates that they have a low permeability.

4.4 Highway #527 Crossing

The stratigraphy at the Highway #527 crossing of Pine Coulee is shown on Drawing No. 15. At this location, Pine Coulee is approximately 220 m in width at the base and 21 m in depth. Bedrock comprises at least three quarters of the valley slope height at this crossing and it outcorps or is thinly covered by colluvium. Clay till overlies the bedrock and extends to the upland ground surface. From within the valley slopes to beneath the valley bottom, the bedrock surface drops some 33.5 m into a U-shaped channel which has become partly infilled with up to 10 m of glaciofluvial sediments extending to the present floodplain level. This deep channel was probably downcut by the late glacial meltwaters which formed Pine Coulee.

Beneath the valley bottom, the bedrock encountered in the boreholes consisted of weathered, silty to clayey shale interbedded with a few thin layers of sandstone which were less than 1 m thick. The overlying glaciofluvial sediments consist of low to medium plastic clays interbedded with a few lenses of silt and sand. These clays are soft to firm at depth but become firm to very stiff near the floodplain surface probably due to dessication.

Beneath the western portion of the valley bottom, the glaciofluvial sediments are interfingered and interlensed with a till-like clay which is probably old slump material. This material was encountered throughout borehole PC-46 and below approximately 6.5 m in borehole PC-45. This till-like clay is mottled medium brown, grey and black and has a reworked blocky structure. It also contains lenses and pockets of sand some of which appear to be reworked, weathered sandstone. Near the floodplain surface, the till-like clay is hard and dessicated while below the water table at approximately 2.5 m, it is soft to very stiff. This clay is underlain by a shale strata which also has a mottled reworked texture near its surface. The depth of this reworking is uncertain due to limited depth of sampling into the bedrock. This old slump, reworked feature probably resulted from a slope movement which occurred when the Pine Coulee channel was being downcut by late glacial meltwaters. At this time, the slopes would have been oversteepened and up to 10 m higher than the present slope. The subsequent infilling of the channel with glaciofluvial sediments probably had a stabilizing effect on the slopes as there is no evidence of recent slope movements within the area.

5.0 Damsite Selection

The selection of the proposed damsite was governed by the existing foundation conditions, the configuration of Pine Coulee and the downstream proximity of the confluence with Willow Creek. At the selected damsite, the foundation conditions are relatively uniform with glaciofluvial clays overlying bedrock at a depth of approximately 14 m. Upstream, the conditions become more complex and less suitable due to the presence of the west to east running buried preglacial valley which crosses beneath Pine Coulee in N 1/2 34-13-28-W4. Approximately 100 m to 200 m upstream of the selected dam centreline, the bedrock surface gradually drops some 5 m to 6 m into what is probably the preglacial Stavely Valley. This valley is infilled by 10 m of dense sand and gravel which is overlain by 11 m of hard, often laminated and slickensided preglacial clays. These preglacial clays are in turn overlain by generally soft to stiff glaciofluvial clays. If the damsite had been located further upstream such that it would have been situated over this preglacial channel, several problems would have had to been overcome. Some of these problems would have included the potential for excess water pressure as was encountered in the basal sands and gravels within boreholes PC-29 and PC-42 as well as the presences of the slickensided preglacial clays which would provide a weak foundation layer.

In addition, at the location of the selected dam centreline, Pine Coulee is at its narrowest and as a result, less earthfill will be required as compared to any other site that could have been chosen either upstream or downstream. Furthermore, since the confluence with the Willow Creek valley is only 400 m to 500 m downstream, any damsite chosen closer to this confluence could have resulted in undesirable seepage passing through the right abutment and daylighting on the north slope of the Willow Creek Valley.

6.0 Diversion Weir Site Selection

In order to divert flow from Willow Creek into Pine Coulee by gravity, the diversion weir location on Willow Creek was selected as being approximately 4 km upstream of the confluence between

Willow Creek and Pine Coulee. The selection of this site is based solely on hydraulic considerations.

7.0 Reservoir Integrity

7.1 Reservoir Watertightness

The watertightness of the Pine Coulee reservoir is discussed in a report prepared by Piteau Engineering Ltd. (1985). The findings presented in this report indicate that the entire reservoir is lined with relatively impermeable sediments comprised of till overlying bedrock along the valley sides, and glaciofluvial or alluvial silts and clays along the valley bottom. As a result, relatively small quantities of seepage losses from the reservoir are expected.

Based on the damsite and hydrogeological investigations performed to date, it would appear that the glaciofluvial clays and silts in the bottom of Pine Coulee will provide a hydraulic separation between the reservoir and the more permeable Stavely Valley sediments. A more detailed investigation is required however, in order to prove the extent and the hydrogeological properties of this buried valley as it has the potential to act as a possible drain from the proposed reservoir.

7.2 Reservoir Slope Stability

In their present condition, the valley slopes along the proposed reservoir appear to be stable with no obvious surficial evidence of recent slope movements discernable on the airphotos (Drawing No. 1). The only possible evidence of a past slope movement is the presence of the reworked material found buried in the valley bottom on the west side of the Highway #527 crossing as described in Section 4.4.

The proposed reservoir can be expected to initiate some slope movements through saturation of the valley slopes and subsequent drawdown. With the reservoir at full supply level, the till and bedrock within the valley slopes will eventually become saturated resulting in some deterioration of shear strength particularly along fissures and joints. As a result, subsequent drawdown may initiate slope movements. Seasonal drawdown of the reservoir will also subject the bedrock to a series of wet-dry cycles further aggravating deterioration, particularly where there are exposed mudstone or shale strata.

Gradual slope degradation due to wave action will produce some beach development around the shoreline of the reservoir. The degree of this degradation will depend on the wave energy environment that could develop within the reservoir.

In the final design, a set-back line encompassing a Restricted Development Area around the proposed reservoir should be established considering the potential and severity of possible future slope movements as well as the magnitude of damage that may occur.

8.0 Foundation Conditions

8.1 Diversion Weir

Along the axis of the diversion weir across Willow Creek, the bedrock profile and the stratigraphy of the alluvium as interpreted from the borehole logs, is presented on Drawing No.13. Between the valley slopes, the length of the weir and its embankments is some 570 m. Across the valley bottom there is a thin layer of low plasticity silty clay covering what appears to be a more or less continuous gravel layer between 2 m and 4 m thick. Below the gravel there is a till deposit.

The foundation conditions across Willow Creek on the proposed alignment hold no special problems with respect to sliding stability or settlement although a sub-surface cut-off is required to control seepage through the gravel layer which exists within the underlying foundation materials.

Downstream of the concrete weir section, a channel will be excavated to return the discharge to the existing river channel.

Borehole PC-6 which lies on the line of this proposed channel indicates that the excavation will be largely in the till layer. Upstream of the concrete weir, Borehole PC-4 shows a veneer of low plasticity silty clay overlying 3 m of silty sand above a low plasticity clay material.

8.2 Diversion Channel

The excavation for the diversion channel will cut through the valley wall immediately upstream of the diversion weir as shown on Drawing No. 21. The excavation slope will have a maximum height of some 26 m. Borehole PC-34 is located close to the point where the excavation will be deepest and its log reveals that most of the slopes will be in till, although a substantial layer of gravel exists near the top of the slope. In the log of borehole PC-34, there is no direct evidence that any deep seated instability has occurred along the Willow Creek valley bank throughout this area.

Just downstream of the entrance to the diversion channel lies the headworks structure (Drawing No. 20). This structure will be a concrete gated structure contained within an embankment crossing the channel. The till foundation within the area is expected to be competent to support this structure.

The profile of the diversion channel which is some 3400 m long is shown on Drawing Nos. 21 and 22. Those boreholes which are close to the alignment of the channel have been projected onto the profile and are shown on the same Drawing. Throughout the upstream half of its length, the channel cross section generally has the invert excavated below natural ground level and the top of the banks are formed by low earth fill embankments. Generally speaking, the invert of the proposed channel lies in or on impervious materials. An exception to this was found at Borehole PC-12 where a gravel deposit was encountered. It is probable that other sand and gravel deposits may also occur elsewhere along the channel. Wherever these deposits are encountered the canal may require lining to prevent excessive water loss.

At approximately the mid-point along the channel, the channel must cross a relatively pronounced valley. The invert of the channel at this crossing will be up to 6 m above the natural ground level. Since the channel banks will be an additional 2.5 m above the channel invert, it may be necessary to excavate some of the silt from the foundation to ensure stability of the side slopes of the canal. The maximum height of fill within this section could be as much as 11 m in the centre of the valley depending upon the depth of stripping required.

Downstream of this valley, the channel enters a 1500 m long section of deep cut. The depth of cut in this area is generally around 7 m although it increases to as much as 11 m at the deepest point. It is anticipated that most of this excavation will be in till although borehole PC-39 encountered a 5.5 m thick layer of gravel at natural ground level. Therefore, it is possible that other sand and gravel layers may also be encountered within the tills during the excavation of the channel. However, since this channel will be in a substantial cut, it may not be necessary to line the channel in order to prevent water loss into the sands and gravels. An exception to this may be near the slopes of Pine Coulee, where it is important that the channel be watertight in order to prevent the possibility of raising the water table in the abutment slopes downstream of the dam. Additional investigations will be required to establish whether or not lining will be required in this area.

8.3 Reservoir Inlet Structure

At the Pine Coulee end of the diversion channel is the reservoir inlet structure. Boreholes PC-21 and PC-22 are closest to the proposed location of the inlet structure. Simplified logs of these holes appear on Drawing No. 22. At the top of the valley wall, borehole PC-21 shows a sequence of till down to a depth of 24 m, or elevation 1032 m. The bed of Pine Coulee is at elevation 1034 m. Below these tills are re-worked tills and preglacial sediments. At a depth of 26 m (elevation 1030 m), slickensides were first encountered. Between a depth of 26 m and 33 m, five undisturbed samples were taken and slickensides were found within four of these samples. The remaining sample contained high plasticity clay lenses and fissures, however, slickensides were not actually reported on the borehole log. The interpretation of these observations is that the slope could be susceptible to a translational type of failure occurring along a pre-sheared plane just below the Pine Coulee valley bottom level.

Near the bottom of the valley wall, borehole PC-22 shows colluvium overlying glaciofluvial sediments, which in turn overlie till and a stiff, low plasticity clay deposit. Beneath the low plasticity clay, a 2.5 m thick gravel layer overlies shale bedrock at elevation 1015 m. The bedrock profile along the entire length of the inlet structure was not identified and this should be established during final design.

8.4 Saddle Dykes

There are in effect two saddle dykes separated by a high spot about 100 m across along the south eastern periphery of the proposed reservoir. The longer dyke is some 2100 m long and reaches a maximum height of about 8.5 m. The other dyke which is further away from the main dam is about 600 m in length and reaches 12.5 m in height. Five boreholes were drilled to verify the foundation conditions along the saddle dykes. Simplified logs of these boreholes are shown on the profile of the saddle dykes on Drawing No. 25. The boreholes indicate that the saddle dykes will be founded on clay till. Shale underlies the till, however, it is at a sufficient depth such that it will not impair the stability of the dykes. No particular problems in respect of the stability, seepage or settlement of the proposed saddle dykes are anticipated.

8.5 Dam Embankment

A stratigraphic section along the proposed dam centreline as inferred from the borehole logs is shown on Drawing No. 33. The dam embankment foundation conditions can generally be divided into three separate sections, namely the river valley, east abutment and west abutment.

8.5.1 River Valley

The preliminary site investigation and laboratory testing along the proposed dam centreline revealed that the foundation materials within the river valley consist of approximately 4.5 m of firm to stiff, low to medium plasticity clay overlying approximately a 4 m thick layer of soft, low to medium plasticity clay. This soft clay layer is in turn underlain by approximately 5.5 m of stiff clay of medium plasticity overlying shale and sandstone bedrock.

8.5.2 East Abutment

On the east abutment, the foundation materials consist of approximately 9 m of brown, hard, low plasticity clay till overlying 6 m of hard, grey, low plasticity clay till. The grey till is underlain by interbedded shale and sandstone bedrock. The shale is moderately fractured to intact. In borehole PC-28, at a depth of 2.5 m, a silty sand layer of approximately 1.5 m in thickness was encountered.

8.5.3 West Abutment

On the west abutment, the foundation materials consist of approximately 12 m of brown, hard, low plasticity clay till overlying 8 m of hard, grey, low plasticity clay till. The grey till is underlain by interbedded shale and sandstone bedrock. The shale is badly fractured to intact. In borehole PC-23, at a depth of 2 m, a silty sand layer of approximately 2 m in thickness was encountered. Several hard silt layers were also encountered at depths between 10 to 16 m.

8.6 Causeway @ Highway #527

A stratigraphic section 40 m north of Highway #527 as inferred from the borehole logs is shown on Drawing No. 39. The foundation conditions across this proposed causeway can generally be divided into three separate sections as the river valley, east abutment and west abutment

8.6.1 River Valley

The preliminary site investigation and laboratory testing revealed that the foundation materials within the river valley consist of approximately 2.5 m of very stiff, low plasticity clay alluvium overlying 7.5 m of very soft to firm, low to medium plasticity clay alluvium. The clay alluvium is underlain by shale bedrock at a depth of approximately 10 m.

8.6.2 East Abutment

On the east abutment, the foundation materials consist of approximately 1 m of medium plasticity clay overlying interbedded shale and sandstone bedrock.

8.6.3 West abutment

On the west abutment, the foundation materials consist of approximately 1.5 m of medium plasticity clay overlying shale bedrock. The shale bedrock is fractured to badly fractured.

9.0 Design Considerations

9.1 Diversion Weir

The proposed concrete diversion weir structure will be constructed on a terrace located on the right bank of the present Willow Creek channel. The excavation required for constructing the structure will reach a maximum depth of 8 m, which is approximately 4 m below the river level. Most of the excavation will be within till although there is a gravel layer which extends to a depth of approximately 2 m below the river level. A cutoff trench may have to be dug through this gravel layer and backfilled with impervious fill in order to keep the excavation dry.

In order to provide a sub-surface cut-off through the gravel layer beneath the proposed dyking adjacent to the concrete weir, it has been assumed that a sheet pile cut-off will be driven across the entire valley to a depth of 5 m which corresponds to approximately 1 m into the impervious material below the gravel. A stratigraphic section of the valley crossing is presented on Drawing No. 13. The gravels were identified in boreholes PC-1 and PC-2 as having a maximum size of 40 mm, while in borehole PC-3 the maximum size was given as 80 mm. In borehole PC-4, there appears to be sand at the same elevation as the gravel was encountered in the other holes.

Once the sheetpile cut-off has been installed beneath the right embankment section, the earthfill for this section can be constructed. Subsequently, after the concrete works for the diversion weir has been completed, Willow Creek can be diverted over the weir and the left embankment constructed. The cofferdam which will be used to divert the river over the weir can also be used to protect the left embankment area during construction.

The maximum height of the dyke embankment will be approximately 10 m. The foundation consists of well drained materials which will provide adequate strength to support the proposed embankments. The proposed embankment cross section is shown on Drawing No. 18. It generally consists of an impervious fill placed with a 3H:1V downstream slope and a 2H:1V upstream slope. The stability of the downstream slope is largely dependent on a drainage blanket which will control the seepage pressures. This drain is to be constructed of free draining gravel. In order to protect the upstream slope of the embankment from wave erosion, 0.7 m of riprap overlying 0.3 m of granular filter is to be provided. The choice of a 2H:1V upstream slope minimizes the amount of riprap required and also permits a simple geometry to be adopted where the crest of the diversion weir is linked to that of the headworks embankment.

To prevent scour of the exit channel for a 12 m length immediately downstream of the diversion weir stilling basin, the channel bed and side slopes shall be protected by 0.7 m of riprap overlying 0.5 m of coarse filter above 0.3 m of fine filter.

In the long term, it is inevitable that some minor slope failures will occur along the left valley wall adjacent to the diversion weir. However, the effects of these failures are anticipated to be minimal since the concrete weir has been located some distance away from the base of the valley wall.

9.2 Diversion Channel

The alignment of the diversion channel has generally been chosen so as to minimize the earthworks costs. One exception is at the location of the new road to Willow Creek Provincial Park where the channel crossing must be at a point where the grade separation between the two is adequate. In roughly the same area, there is a sandstone outcrop which runs along the east side of a shallow dry coulee. This sandstone outcrop has been avoided since it would be difficult to excavate the hard rock and also it would probably require that the channel be lined in order to prevent excessive water loss. A third exception has to do with the advisability of keeping the channel well away from the slopes of the Willow Creek and Pine Coulee valleys. In this way, leakage from the channel will not raise the water table in these slopes and as a result, impair their stability.

It has been assumed that the proposed channel berms will be constructed from the material excavated from the channel and that no additional borrow will be required.

A very large cut section is required at the upstream end of the diversion channel. This cut will be entirely in a till deposit which lies above the natural water table. Preliminary slopes of 3H:1V have been chosen for this 26 m deep cut. The surface of these slopes should be topsoiled and seeded in order to prevent runoff erosion.

To prevent scour of the diversion channel, the side slopes and channel bed shall be protected by 0.2 m of gravel lining.

9.3 Reservoir Inlet Structure

The proposed inlet chute will have a chute slope inclination of 4H:1V which approximates the existing natural slope. Underdrainage is provided by a 1.0 m thick coarse gravel layer which is separated from the finer natural soils by a fine filter layer. Perforated pipe drains have been incorporated into the coarse gravel layer to commute seepage water from the gravel to discharge pipes buried along the sides of the chute as shown on Drawing No. 23. These drains will terminate into the gravel and rockfill zone at the bottom of the chute. To prevent downslope movement of the gravel, cross-walls are provided at intervals along the chute.

The strength parameters used for evaluating the stability of the chute slope are presented in Table 3. The minimum factor of safety calculated for the proposed chute slope under the most severe drawdown conditions was 1.5. This analysis is presented on

Drawing No. 24. It is to be noted that the minimum drawdown level used for this analysis was elevation 1044 m which is the minimum permanent pool level selected for the reservoir.

9.4 Saddle Dyke

The selected cross section for the proposed saddle dyke is shown on Drawing No. 25. The proposed dyke will be a homogeneous embankment with a blanket drain at the toe to control seepage pressures beneath the downstream slope. A shallow cut-off trench has been provided to reduce seepage through any sand seams close to the foundation surface.

The stability of the highest section of dyke has been analysed using similar strength parameters as presented in TAble 3. The foundation investigation indicated that the foundation materials below the dyke will be primarily till and therefore, a uniform till foundation layer was used in the analyses. The results of the stability analyses are presented on Drawing Nos. 27, 28 and 29. For all stability conditions evaluated, the calculated factor of safety exceeded 1.5.

To prevent scour due to wave action, the entire upstream slope of the north-south section of the saddle dyke shall be protected by 0.80 m of riprap overlying 0.5 m of coarse filter above 0.2 m of fine filter while the east-west section of the dyke shall be

TABLE 3

Soil Strength Parameters Used for Design

Soil Type	c' (KPa)	ϕ' (Degree)	Unit Weight (kg/m^3)
Foundation Materials			
CL (Clayey Alluvium)	7.0	27	1,960
CL (Surficial Till)	10.0	25	2,100
CL (Clayey Alluvium)	7.0	26	1,960
Shale Bedrock	20.00	18	2,100
Fill Material			
CL (Till) Fill (@ +2% O.M.C.)	3.5	28	2,000
Granular Materials	0	32	2,080

protected by 0.6 m of riprap overlying 0.4 m of coarse filter above 0.2 m fine filter.

9.5 Dam Embankment

9.5.1 General

The proposed dam will be a homogeneous earthfill approximately 21 m in height and 450 m in length along its crest. The configuration of the upstream and downstream slopes as well as the toe berms to be provided on both sides of the dam are shown on Drawing No. 31.

9.5.2 Slope Stability Analysis

The foundation conditions within the central portion of the valley were considered to be critical and thus they were incorporated in the typical cross section which was used for the stability analyses.

A summary of the strength parameters used for the stability analyses is presented in Table 3. The strength parameters used for the total stress and effective stress analyses are also presented on Drawing Nos. 34, 35 and 36.

The appropriate strength parameters to be used for the shale and sandstone bedrock materials were very difficult to assess since these parameters are dependant upon the degree of fracture and the amount of slickensiding that exists within the material. Due to the preliminary nature of the study, the effects of fracturing and slickensiding were not investigated in detail at this time but rather average shear strength parameters of $\phi = 18^{\circ}$ and $c' = 20$ KPa were assumed for the bedrock materials.

The pore pressure response that will occur within the foundation soils due to embankment loading is also difficult to assess. However, based on the available foundation soil data, the following assumptions were made for the purpose of performing the preliminary stability analysis.

1) The embankment will be constructed in two stages with each stage being constructed during a single construction season. The first stage will be constructed to El. 1040.3 while the second stage will be constructed to the dam crest elevation of 1054.4.

2) The embankment fill will be placed at 2% wet of the optimum moisture content and will have a pore pressure ratio throughout the construction season of $r_u = 0.25$.

3) The excess pore water pressures generated within the foundation clay layers due to embankment loading were assumed to be $r_u = 0.7$ for both stages of construction. In addition, it was assumed that all of the excess pore water pressure that would be induced by the Stage I loading would be completely dissipated prior to the commencement of Stage II construction. To facilitate this pore pressure dissipation, wick drains are being proposed as described in a subsequent section.

For the stability analysis, the maximum embankment fill section as shown on Drawing No. 34 was evaluated. This section located within the central portion of the river valley was considered representative of the most critical condition along the dam centreline. The Simplified Janbu's method of analyses was used for all the dam embankment slope stability analyses. The eventual dam cross section which evolved as a result of the various stability analyses that were completed is presented on Drawing No. 31.

a) Stability of the Dam Embankment Slopes During Construction

Due to the existence of a soft clay layer near the mid-depth of the valley glaciofluvial deposits, total stress analyses were considered to be the most appropriate for evaluating the end of construction conditions. The total strength parameters that were assumed for the stability analyses were established by evaluating the unconfined compressive strength of undisturbed samples taken during the foundation investigation. The strength parameters used were as follows: $\phi = 18^\circ$ and c = 10 KPa for the low plasticity clay till fill; $\phi = 0$ and c = 12 KPa for the soft, low to medium plasticity clay; $\phi = 0$ and c = 50 KPa for the upper stiff, low to medium plasticity clay; and $\phi = 0$ and c = 75 KPa for the lower stiff, medium plasticity clay. In addition, it was assumed that due to the provision of wick drains beneath the upstream and downstream slopes of the dam embankment, the strength of the soft clay layer would increase as a result of consolidation prior to the commencement of the Stage II construction. Therefore, the increased strength parameters that were used for the soft clay layer to evaluating the stability of the Stage II construction were $\phi = 0$ and $c = 34$ KPa.

For the Stage I construction, an embankment fill height of El. 1040.3 was selected. Assuming the end of construction condition and using the assumed total strength parameters as previously described, a factor of safety of 1.38 was calculated for the proposed 8H:1V upstream and downstream slopes. The location of the critical failure surface is shown on Drawing No. 35. The stability of these slopes was also evaluated using effective stress analyses. Assuming the effective strength and pore water pressure parameters as previously described, the calculated factor of safety for the 8H:1V slopes was in excess of 1.50.

For the end of Stage II construction condition and again assuming the total strength parameters as previously described, a factor of safety of 1.33 was calculated for the proposed upstream and downstream slopes. The location of the critical failure surface is shown on Drawing No. 36. For the effective stress analyses using the assumed effective strength and pore water pressure parameters as previously described, the calculated factor of safety for the proposed slopes was again in excess of 1.50.

b) Stability of the Dam Embankment Upstream Slope During Drawdown

The critical stability condition for the proposed upstream slope of the dam was found to be the end of construction condition. However, the stability of the upstream slope during maximum drawdown of the reservoir from the FSL elevation of 1052.5 to elevation 1044.0 was also evaluated in some detail. Using effective stress analyses, the critical factor of safety for the drawdown condition was calculated to be 1.60.

c) Long Term Stability of the Dam Embankment Downstream Slope

The critical stability condition for the proposed downstream slope of the dam was again found to be the end of construction condition. However, the stability of the downstream slope assuming the steady stage seepage condition with the upstream reservoir water level at the FSL elevation of 1052.5, was also evaluated. For this analyses, it was assumed that all the seepage passing through the dam embankment would be intercepted by either the chimney drain or the horizontal drainage blanket. The critical factor of safety for this condition was calculated to be considerably in excess of 1.50.

e) Stability of the Natural Dam Abutment Slopes

The overall stability of the existing valley slopes adjacent to the proposed dam embankment after having been subjected to the influence of the reservoir impoundment was also analyzed. A cross section of the east bank approximately 100 m upstream of the dam centreline was chosen as a typical valley bank section for this analysis. The stratigraphic section used for the analysis is shown on Drawing No. 37. A summary of the strength parameters used for the various materials present within the valley bank was previously included in Table 3.

The existing valley slopes in the vicinity of the east abutment are inclined at approximately 3H:IV to 4H:IV. The critical condition for the stability of these slopes will be the reservoir drawdown condition. A critical factor of safety of 1.36 was calculated for a 3H:IV slope assuming complete reservoir drawdown from the FSL elevation of 1052.5 to elevation 1044.0.

9.5.3 Dam Embankment Settlement

The preliminary foundation investigation indicated that along the dam centreline, approximately 14 m of glaciofluvial clay overlies shale and sandstone bedrock. Since the abutments consist of clay till overlying relatively shallow bedrock, some differential settlement between the abutments and the valley bottom portion of the dam embankment is anticipated.

A summary of the consolidation test results used as a basis for the settlement calculations is shown in Table 4. Based on these test results, a maximum settlement of 0.70 m has been estimated for the central river valley portion of the dam embankment. However, due to the installation of the wick drains as well as the proposed stage construction, it is anticipated that as much as 40 percent or 0.30 m of this settlement will occur during the actual construction period. Furthermore, in order to minimize potential cracking due to the remaining post construction differential settlement, it has been assumed that the impervious fill material used to construct the dam will be placed at 2% wet of the Standard Proctor optimum moisture content.

TABLE 4

Summary of Consolidation Tests

Borehole Number	Depth (m)	Soil Classification	Moisture Content (%)	Atterberg Limits w_L	w_P	I_P	Grain Size Sand %	Silt %	Clay %	Initial Void Ratio e_0	Approximate Overburden Pressure Po' (kPa)	Pre-Consolidation Pressure Pc' (kPa)	Compression Index C_c	Coefficient of Consolidation c_v (cm²/sec)	Coefficient of Permeability k (cm/sec)
[illegible]	6.10-6.55	CL	18.1	[illegible]	11.[illegible]	19.[illegible]	<1	79	21	0.69	73.5	215	0.08	0.99-1.1x10^{-1}	1.5-3.7x10^{-7}
PC-36	6.55-6.95	CL	25.5	27	15.[illegible]	11.6	3	71	26	0.60	62.1	190	0.135	1.83-2.0x10^{-2}	0.7-1.7x10^{-7}
PC-27	6.10-6.40	CL	17.45	23.0	12.1	10.9	<1	78	21	0.67	96.20	76	0.079	1.0-2.9x10^{-2}	0.3-2.0x10^{-7}
PC-29	1.50-1.90	CH	22.61	56.7	18.5	38.2	1	50	49	0.62	14.71	730	0.169	1.6-4.9x10^{-4}	0.4-1.5x10^{-8}
PC-42	6.10-6.55	CI	28.23	32.6	14.6	18.0	6	59	35	0.75	59.80	97	0.167	2.7-3.6x10^{-3}	1.0-4.0x10^{-8}
PC-42	12.20-12.5	CI	26.32	39.5	18.5	21.0	1	43	56	0.65	119.7	350	0.127	7.9-8.5x10^{-3}	2.3-4.0x10^{-8}
PC-43	6.10-6.45	CI	29.43	39.8	17.5	22.3	12	40	48	0.79	96.7	76	0.178	5.0-11.0x10^{-4}	0.1-3.0x10^{-8}
PC-45	1.50-1.85	CI	22.30	46.6	17.1	29.5	1	50	49	0.61	31.0	310	0.133	8.6-9.0x10^{-3}	2.0-5.0x10^{-10}
PC-45	3.05-3.55	CL	29.03	28.7	17.8	10.9	2	75	23	0.80	43.0	120	0.131	8.0-10x10^{-3}	3.3-12.5x10^{-8}
PC-56	6.55-6.95	CI	26.42	39.6	19.6	20.0	26	31	43	0.65	64.0	110	0.102	4.3-7.0x10^{-4}	3.0-10.0x10^{-8}
PC-55	1.50-1.95	CI	29.23	42.4	20.0	22.4	3	55	42	0.78	29.4	300	0.213	6.0-7.0x10^{-4}	3.0-9.0x10^{-8}

9.5.4 Wick Drains

In order to adequately dissipate the excess pore water pressures that are anticipated to develop within the soft clay layer which exists within the river valley, wick drains are required beneath both the upstream and downstream slopes as shown on Drawing No. 31. If the wick drains were not provided, much flatter embankment slopes would be required in order for them to be stable especially during dam construction.

The proposed wick drains have been arranged in a triangular array with a 3 m spacing across a 230 m width of the valley bottom. The depth to which the drains are to be installed will be approximately 8 m. while the width of the proposed installations will be approximately 120 m beneath both the upstream and downstream embankment slopes. The wick drains will not be installed within a 40 m wide section beneath the central core of the embankment in order to adequately restrict the flow of seepage beneath the dam. It should be noted that in order to more precisely determine the required spacing of these wick drains, extensive field permeability testing will have to be performed during the final design.

9.5.5 Seepage Considerations

Seepage flow through the dam embankment must be adequately controlled such that no long term internal erosion due to piping will occur. In order to intercept the seepage that will pass through the dam embankment, an inclined chimney drain connected to a downstream horizontal drainage blanket has been provided. The downstream horizontal drainage blanket will be founded directly above the foundation clay deposit at approximately elevation 1033.30. It will be extended across the entire river valley and up to the valley banks to elevation 1041.3 on each abutment. The drainage blanket will commute the seepage water collected by the chimney drain as well as discharge from the wick drains. The drainage blanket will terminate into a discharge channel at the downstream toe of the dam.

9.5.6 Scour Protection

To prevent scour due to wave action, the 4H:IV slope on the upper portion of the upstream slope of the dam embankment shall be protected from erosion by 0.6 m of riprap overlying 0.4 m of coarse filter above 0.2 m of fine filter. The riprap protection shall extend from the dam crest level down to the top of the berm at elevation 1043.3. On the lower portion of the upstream slope where the slope is 8H:IV, the slope as well as the top of the berm shall be protected by 0.3 m of pitrun gravel. The lower portion of the dam embankment does not require as extensive an erosion protection as the upper portion since it will be below the minimum permanent pool level and thus less susceptible to wave attack.

To prevent piping of the internal drainage blanket materials, the downstream toe of the dam embankment shall be protected by a 0.5 m thick layer of riprap extending from the toe of the dam up to 0.5 m above the top of the horizontal drainage blanket.

9.5.7 Impervious Borrow Materials

The preliminary site investigation and laboratory testing has revealed that the surficial material overlying virtually all of the uplands within the vicinity of the damsite is generally low plasticity glacial clay till. The laboratory test results indicate that this low plastic clay till material has a relatively high silt content which is generally not considered to be desirable from a dam construction point of view although it can be used if proper design precautions are observed. For this preliminary design, it was assumed that the low plasticity clay would be used as the fill material, however, during final design, additional site investigations will be required to attempt to locate a more suitable embankment fill material.

It is estimated that the total quantity of impervious material required for the construction of the dam will be approximately 1.2 million m^3. It is anticipated that all of this material could be obtained from the borrow area located north of the damsite as shown on Drawing No. 17.

An evaluation of the impervious materials available within the proposed borrow area indicated that the in-situ moisture content of the material ranges from 9.5 to 11% with an average of 10% while the optimum moisture content ranges from 11 to 13% with an average of 12%. Since the intent is to place the impervious materials wet of optimum, it is anticipated that the addition of water to the in-situ material will be required.

9.5.8 Instrumentation

An instrumentation program is proposed to observe the behaviour of the dam foundation, embankment and abutments both during construction as well as during long term operation.

The overall instrumentation program will provide information which will be used to control the rate of construction, verify design parameters and monitor the critical elements contributing to the satisfactory long term performance of the dam embankment.

The proposed program shall be capable of monitoring the following:

1. Pore water pressures within the foundation and embankment during construction as well as after impoundment of the reservoir.
2. Settlement and rate of consolidation of the dam embankment and foundation materials.
3. Lateral spreading of the dam embankment at the foundation contact.

4. Stability of the abutments during construction as well as during the initial filling and subsequent drawdown of the reservoir.

The proposed instrumentation shall include a network of piezometers, settlement gauges, slope indicators as well as slope and dam crest survey monuments for surface displacement measurements. Details of the types of instrumentation to be used, the locations of the installations and other pertinent installation details shall be established as part of the final design.

9.5.9 Low Level Outlet Structure

a) General

The low level outlet structure shall consist of a 326 m long cast-in-place reinforced concrete conduit with internal dimensions of 1.5 m in width and 2.5 m in height on the upstream of the gate well and 1.5 m in width and 2.13 m in height on the downstream of the gate well as shown on Drawing No. 32. The purpose of the low level outlet is to divert the river flow during dam construction, regulate the release of the stored water and control the flood flows originated within Pine Coulee.

In order to avoid placing the conduit over the soft clay valley foundation materials, it has been located within the east abutment where the foundation material are much more competent and less prone to differential settlement.

The existing east abutment slope in the vicinity of the dam centreline varies from 3H:IV to 4H:IV. Construction of the conduit will necessitate a rather extensive excavation into this abutment slope. For preliminary design purposes, it was assumed that all of the temporary excavation slopes required for constructing the proposed conduit will be stable at 1H:IV.

To prevent scour of the exit channel downstream of the low level outlet basin, the channel bed and side slopes for a 10 m length shall be protected by 0.5 m of riprap overlying 0.3 m of coarse filter above 0.2 m of fine filter.

b) Conduit Settlement

The proposed outlet structure will be a cast-in-place reinforced concrete conduit with sufficient joints between adjoining sections to accommodate lateral displacement of the dam embankment as well as differential settlement along the conduit. A detailed settlement profile along the conduit was not established, however, the maximum settlement beneath the dam centreline was estimated to be in the order of approximately 0.23 m. The proposed jointing system can readily be designed to accommodate this magnitude of settlement.

9.6 Causeway @ Highway #527

9.6.1 General

The proposed causeway will be homogeneous earthfill as shown on Drawing No. 38. Eventually the causeway will be paved in accordance with Alberta Transportation standards.

The existing Highway #527 crossing of Pine Coulee was raised to a minimum elevation of 1049.0 during this past year by Alberta Transportation. Therefore in order to construct the causeway to the designed grade of 1054.2, a maximum fill height of 5.2 m will be required on top of the existing highway. Furthermore, in order to convey the ponded water within the reservoir across the causeway, the existing S.P.C.S.P. culvert will have to be extended and headwalls provided.

9.6.2 Slope Stability Analysis

Based on the stratigraphy as shown on Drawing No. 39 established for a section some 40 m north of the existing highway, the foundation materials within the river valley consist of approximately 2.5 m of very stiff, low plasticity clay overlying 7.5 m of very soft to firm, low to medium plasticity clay. The soft to firm clay layer is underlain by shale bedrock.

The foundation stratigraphy beneath the proposed highway crossing is very similar to that identified beneath the main dam embankment and therefore, many of the design requirements which were applicable for the dam such as stage construction and the use of wick drains are also applicable to the highway crossing. Therefore, the proposed highway embankment will be constructed in two stages with each stage being constructed within a single construction season. The first stage is to be constructed to elevation 1043.5 while the second stage will be constructed to the designed grade elevation of 1054.2.

Based on the preliminary foundation investigation, the following assumptions were made for the purposes of performing the preliminary stability analysis.

1. For the stability analyses, the maximum fill section as shown on Drawing No. 38 was evaluated. This section which is representative of the central portion of the river valley, was considered to be the most critical. The Simplified Janbu's method of analyses was used for all the causeway slope stability analyses. The eventual causeway cross section which evolved as a result of the various stability analyses that were completed is presented on Drawing No. 38.

2. The embankment fill will be placed at 2% wet of the optimum moisture content and will have a pore pressure ratio throughout the construction season of $r_u = 0.25$.

3. The excess pore water pressures generated within the foundation clay layers due to embankment loading were assumed to be $r_u = 0.7$ for both stages of construction. In

addition, it was assumed that all of the excess pore water pressure that would be induced by the Stage I loading would be completely dissipated prior to the commencement of Stage II construction. To facilitate this pore pressure dissipation, wick drains are being proposed as described in a subsequent section.

4. The strength parameters that were used for the stability analyses were similar to those used for the main dam embankment. A summary of these strength parameters was previously presented in Table 3. The strength parameters used for the total stress and effective stress analyses are also presented on Drawing Nos. 40, 41 and 42.

a) Stability of the Causeway Slopes During Construction

Due to the existence of a soft clay layer located 2.5 m below the valley floor, total stress analyses were considered to be most appropriate for evaluating the end of construction conditions. The assumed strength parameters for the total stress analyses were as follows: $\phi = 18^{\circ}$ and $c = 10$ KPa for the low plasticity clay till fill; $\phi = 0$ and $c = 50$ KPa for the very stiff clay; and $\phi = 0$ and $c = 20$ KPa for the soft to firm clay. In addition, it was assumed that due to the provision of wick drains beneath the upstream and downstream slopes of the causeway, the strength of the soft clay layer would increase as a result of consolidation prior to the commencement of the Stage II construction. Therefore, the increased strength parameters that were used for the soft clay layer to evaluate the stability of the Stage II construction were $\phi = 0$ and $c = 35$ KPa.

For the Stage I construction, an embankment fill height of elevation 1043.5 was selected. Assuming the end of construction condition and using the assumed total strength parameters as previously described, a factor of safety of 1.69 was calculated for the proposed 8H:1V upstream and downstream slopes. The location of the critical failure surface is shown on Drawing No. 41. The stability of these slopes was also evaluated using effective stress analyses. Assuming the effective strength and pore water pressure parameters as previously described, the calculated factor of safety for the 8H:1V slopes was in excess of 1.50.

For the end of Stage II construction condition, and again assuming the total strength parameters as previously described, a factor of safety of 1.28 was calculated for the proposed upstream and downstream slopes. The location of the critical failure surface is shown on Drawing No. 42. For the effective stress analysis using the assumed effective strength and pore water pressure parameters as previously described, the calculated factor of safety for the proposed slopes was again in excess of 1.50.

b) Stability of the Causeway Slopes During Drawdown

The critical stability condition for the proposed upstream and downstream slopes of the causeway was found to be the end of construction condition. However, the stability of the upstream and downstream slopes during maximum drawdown of the reservoir from the FSL elevation of 1052.5 to elevation 1044.0 was also evaluated in some detail. Using the effective stress analyses, the critical factor of safety for the drawdown condition was calculated to be in excess of 1.50.

9.6.3 Wick Drains

In order to adequately dissipate the excess pore water pressures that are anticipated to develop within the soft clay layer which exists within the river valley, wick drains are required beneath both the upstream and downstream slopes as shown on Drawing No. 38. If the wick drains were not provided, much flatter embankment slopes would be required in order for them to be stable especially during dam construction.

The proposed wick drains have been arranged in a triangular array with a 3 m spacing across a 220 m width of the valley bottom. The depth to which the drains are to be installed will be approximately 10 m while the width of the proposed installations will be approximately 120 m beneath both the upstream and downstream embankment slopes. No wick drains will be installed beneath the existing highway embankment. It should be noted that in order to more precisely determine the required spacing of these wick drains extensive field permeability testing will have to be performed.

9.6.4 Scour Protection

To prevent scour due to wave action, the 4H:IV slope on the upper portion of both the upstream and downstream slopes of the causeway shall be protected from erosion by 0.6 m of riprap overlying 0.4 m of coarse filter above 0.2 m of fine filter. The riprap protection shall extend from the dam crest level down to the top of the berm at elevation 1043.5. On the lower portion of both the upstream and downstream slopes where slope is 8H:IV, the slope as well as the top of the berm shall be protected by 0.3 m of pitrun gravel. The lower portion of the causeway does not require as extensive an erosion protection as the upper portion since it will be below the minimum permanent pool level and thus less susceptible to wave attack.

9.6.5 Impervious Borrow Materials

The preliminary site investigation and laboratory testing has revealed that the surficial material overlying the upland within the vicinity of the causeway is generally low plasticity glacial clay till. The laboratory test results indicate that this low plastic clay till material has a relatively high silt content which is generally not considered to be desirable from an embankment construction point of view although it can be used if proper design precautions are observed. For this preliminary design, it was assumed that the low plasticity clay would be used as the fill material, however, during final design, additional site investigations will be required to attempt to locate a more suitable embankment fill material.

It is estimated that the total quantity of impervious material required for the construction of the causeway will be approximately 314,000 m^3. It is anticipated that all of these materials could be obtained from the borrow area located northwest of the causeway as shown on Drawing No. 38.

An evaluation of the impervious materials available within the proposed borrow area indicated that the in-situ moisture content of the material ranges from 10 to 12% with an average of 11% while the optimum moisture content is approximately 13%. Since the intention is to place the impervious materials wet of optimum, it is anticipated that the addition of water to the in-situ material will be required.

9.7 East-West Road Causeway @ Northern Boundary of 33-14-28-W4

As shown on Drawing No. 43, the existing road embankment at this crossing will have to be raised 1.5 m to an elevation of 1053.9. No site investigation was performed in this location, however, for preliminary cost estimate purposes, 4H:IV side slopes for the embankment fill were assumed. Both road side slopes will require erosion protection consisting of 0.3 m of rock riprap overlying 0.5 m of granular filter material.

The existing road culvert will be replaced with a gated C.S.P. control structure so as to maintain a miniumum flood level upstream.

10. Availability of Granular Materials and Rock Riprap

A large quantity of riprap material will be required for constructing the various component of this project. Unfortunately, however, sources of riprap are somewhat scarce in this part of the Province. For preliminary cost estimate purposes, it was assumed that the source of riprap material would be the Rocky Mountain foothills with a hauling distance of approximately 160 km.

Large quantities of granular material will also be required. A very preliminary search for granular materials was undertaken. Several sources of granular material with hauling distances ranging from 7 to 80 km were identified.

During final design, a much more extensive investigation program will be required to determine the location, quality and quantity of riprap and granular material available within this area.

11. Conclusions and Recommendations

A preliminary study has been completed for a proposed off-stream storage reservoir within Pine Coulee along with the associated diversion works from Willow Creek. The construction of the proposed earthfill dam across the downstream end of Pine Coulee to impound a reservoir of approximately 50.3 million m^3 is geotechnically feasible.

A summary of the major geotechnical aspects associated with the preliminary design of this project is as follows:

1. The proposed dam embankment design cross section is as shown on Drawing No. 31. The proposed dam is primarily a homogeneous earthfill structure with an internal drainage system.

2. The proposed causeway at Highway #527 design cross section is as shown on Drawing No. 38. This causeway will also be homogeneous earthfill structure.

3. Under the dam centreline up to 14 m of glaciofluvial sediments infill a U-shaped bedrock channel. These glaciofluvial sediments primarily consist of compressible low to high plasticity clay. The foundation materials within both abutments generally consist of low plasticity clay till overlying bedrock.

4. Under the proposed causeway at Highway #527, the valley is also infilled with up to 10 m of low to high plasticity clay. The foundation materials within the abutments generally consist of a thin layer of low plasticity clay overlying bedrock.

5. It is anticipated that some differential settlement between the valley bottom and the abutments will occur for both the dam embankment and causeway at Highway #527 crossing. In order to minimize potential cracking due to this differential settlement, the impervious fill material used to construct the dam shall be made more ductile by compacting at water contents ranging between 100 and 115 percent of the Standard Proctor optimum water content.

6. Due to the existence of a soft clay layer beneath the dam embankment and causeway at Highway #527, wick drains shall be installed beneath the embankment slopes in order to provide adequate dissipation of pore pressures and in this way, ensure stability of the embankment slopes during construction.

7. To control seepage within the dam embankment, an internal chimney drain and horizontal drainage blanket are recommended. Horizontal Drainage blankets have also been incorporated into the causeway at Highway #527 crossing.

8. Two impervious borrow areas have been identified within the project area. One is located on the upland west of Pine Coulee approximately one kilometer north of the proposed damsite. The other is located on the upland adjacent to the west abutment of the proposed causeway at Highway #527. The materials within these borrow areas is a low plasticity clay till with relatively high silt content. During final design, additional site investigations will be required to attempt to locate a more suitable embankment fill material.

9. To prevent scour due to wave action, the upstream slope of the dam embankment as well as both side slopes of the causeway at Highway #527 above the permanent pool elevation of 1044.0 shall be protected by riprap.

10. In order to locate the low level outlet conduit away from the soft clay deposit within the valley bottom, it shall be constructed within the east abutment as shown on Drawing No. 30. By avoiding the placement of the conduit over the soft valley bottom clays, the amount of differential settlement along the conduit can be minimized.

11. The configuration of the diversion weir on Willow Creek and the Pine Coulee Reservoir inlet are shown on Drawing Nos. 19 and 23 respectively. These structures shall be provided with complete under-drainage systems in order to prevent the build-up of excessive hydrostatic pressure during operation. The upstream slope of the diversion dyking shall be protected from wave action by installing riprap.

12. The alignment of the proposed diversion channel has been selected so as to minimize the earthworks costs. Some minor lining may be required in certain isolated areas in order to minimize excessive seepage losses.

13. Saddle dyking is required along the southeastern periphery of the proposed reservoir. The reservoir facing slope of this dyking shall be protected from wave action by installing riprap.

14. The watertightness of the Pine Coulee reservoir is discussed in a report entitled "Hydrogeological Assessment of the Proposed Pine Coulee Reservoir" prepared by Piteau Engineering Ltd. in October 1985. The report indicates that water losses from the reservoir would appear to be relatively insignificant.

12. Additional Study Requirements for Final Design

In completing the preliminary design, some specific areas requiring additional study have been identified.

1. In the final design stage, a detailed site investigation and a comprehensive laboratory testing program is required for the proposed dam embankment, the causeway at Highway #527, the diversion weir and the reservoir inlet structure to confirm the strength parameters used for the foundation materials in the preliminary analysis.

2. A detailed evaluation of the materials to be excavated from the diversion channel in the vicinity of the proposed dam is required in order to confirm the overall suitability of the material for use as dam embankment fill.

3. The preliminary site investigations have indicated that most of the fill within the project area is of low plasticity with a relatively high silt content. An extensive impervious borrow material search will be required to identify the most suitable clay till fill material within the area.

4. A large quantity of riprap will be required for this project. It was assumed for cost estimating purposes that the source of rock to be used would be located within the Rocky Mountain foothills. The availability and suitability of the riprap within the foothills will have to be confirmed during final design. In addition, more extensive investigation should be undertaken to locate closer sources of riprap such that the cost of procuring these materials can be minimized.

   Large quantities of granular material will also be required. A very preliminary search for granular materials was undertaken. Several sources of granular materials within a hauling distance of approximately 80 km were identified. During final design, a much more extensive investigation program will be required to determine the location, quality and quantity of material available within this area.

5. A detailed study is required to establish the installation and operational details as well as the costs associated with the installation of the proposed wick drains.

6. A detailed cost comparison relative to constructing a bridge across Pine Coulee at the location of the proposed Highway #527 causeway should be performed. A detailed analysis may indicate that a bridge crossing is slightly more economical than the proposed embankment fill.

7. It would appear that old slumping materials may exist beneath the west portion of the valley in the vicinity of the causeway at Highway #527. A more detailed site investigation is required to better define this slump material.

8. The Stavely buried channel should be more accurately delineated to ensure that no hydraulic connection exists between it and the proposed reservoir.

9. A more detailed investigation is required around the periphery of the proposed reservoir to determine if a setback line or a restricted development area is required.

13. Cost Estimate

The following cost estimate has been prepared by the Geotechnical and Design Branches of the Design and Construction Division for the overall Pine Coulee Reservoir Project.

WILLOW CREEK PROJECT

PINE COULEE RESERVOIR

COST SUMMARY

A)	Site Services	$ 980,000
B)	Diversion Weir	$ 3,450,000
C)	Headgates	$ 250,000
D)	Diversion Channel	$ 2,182,000
E)	Reservoir Inlet	$ 327,000
F)	Dam Embankment	$ 8,922,000
G)	Outlet Works	$ 1,420,000
H)	Saddle Dyke	$ 4,410,000
I)	Causeway @ Highway #527	$ 3,580,000
J)	Causeway @ E-W Road (N.Bdy. of 33-14-28-w4)	$ 572,000
K)	Road Relocation	$ 700,000
L)	Reservoir	$ 2,700,000
	Total	$ 29,493,000
	Say	$ 29,500,000

A detailed description itemizing each of the costs presented in the preceding summary has been completed and has been included on the following pages. The total estimated cost of $29.5 million for the overall project includes a 15% allowance for contingencies but does not include any provision for engineering costs.

COST ITEM

A) Site Services

1)	Performance Bond	L.S.	$ 80,000
2)	Labour & Materials Bond	L.S.	$ 40,000
3)	Construction Camp & Office	L.S.	$ 200,000
4)	Construction Roads	5 km @ $25,000	$ 125,000
5)	Powerlines	10 km @ $20,000	$ 200,000
6)	Utilities	L.S.	$ 50,000
7)	Surface Drainage	L.S.	$ 50,000
8)	Landscaping	L.S.	$ 50,000
9)	Miscellaneous		$ 50,000
			$ 845,000
		x 1.15	$ 971,750
		Say	$ 980,000

B) **Diversion Weir**

1)	Mobilization & Demobilization	L.S.	$ 100,000
2)	Care of Water	L.S.	$ 150,000
3)	Common Excavation	228,000 m^3 @ $1.6	$ 364,800
4)	Structural Excavation	1,000 m^3 @ $3.0	$ 3,000
5)	Compacted Impervious Fill	53,000 m^3 @ $1.00	$ 53,000
6)	Tamped Impervious Backfill	1,000 m^3 @ $5.00	$ 5,000
7)	Riprap	6,700 m^3 @ $50.00	$ 335,000
8)	Coarse Filter	2,300 ,^{3} @ $15.00	$ 34,500
9)	Fine Filter	1,100 m^3 @ $18.00	$ 19,800
10)	Drain Gravel	4,400 m^3 @ $20.00	$ 88,000
11)	Topsoil & Seeding	4.7 Ha @ $7000.0	$ 32,900
12)	Sheet Piling	500 m^2 @ 200.0	$ 100,000
13)	Lean Concrete	140 m^3 @ 200.0	$ 28,000
14)	Reinforced Concrete	4,100 m^3 @ $350.0	$1,435,000
15)	Gate & Hoist, 5000 x 4000 mm Radial Gate	1 Unit @ $100,000	$ 100,000
16)	Fish Pass	L.S.	$ 50,000
17)	Miscellaneous		$ 100,000
			$2,999,000
		x 1.15	$3,448,850
		Say	$3,450,000

C) **Headgates**

1)	Structural Excavation	300 m^3 @ \$3.0	\$ 900
2)	Compacted Impervious Fill	3900 m^3 @ \$1.00	\$ 3,900
3)	Tamped Impervious Backfill	1300 m^3 @ \$5.00	\$ 6,500
4)	Riprap	700 m^3 @ 50.0	\$ 35,000
5)	Coarse Filter	400 m^3 @ \$15.00	\$ 6,000
6)	Fine Filter	200 m^3 @ \$18.00	\$ 3,600
7)	Drain Gravel	200 m^3 @ \$20.00	\$ 4,000
8)	Road Gravel	10 m^3 @ \$10.0	\$ 100
9)	Lean Concrete	10 m^3 @ \$200.0	\$ 2,000
10)	Reinforced Concrete	250 m^3 @ \$400.0	\$100,000
11)	Gates & Hoists, 72" x 96" Sluice Gate	3 Units @ \$10,000	\$ 30,000
12)	Miscellaneous		\$ 25,000
			\$217,000
		x 1.15	\$249,550
		Say	\$250,000

D) **Diversion Channel**

1)	Common Excavation	790,500 m^3 @ \$1.60	\$1,264,800
2)	Compacted Impervious Fill	82,000 m^3 @ 1.00	\$ 82,000
3)	Gravel Liner	16,000 m^3 @ \$10.00	\$ 160,000
4)	Road Gravel	1,600 m^3 @ \$10.00	\$ 16,000
5)	Topsoil & Seeding	3.50 Ha @ \$7,000	\$ 24,500
6)	Cross Drainage	2 Units @ \$50,000	\$ 100,000
7)	Bridge	200 m^2 @ \$1,000	\$ 200,000
8)	Fencing		\$ 20,000
9)	Miscellaneous		\$ 30,000
			\$1,897,300
		x 1.15	\$2,181,895
		Say	\$2,182,000

E) Reservoir Inlet

1)	Structural Excavation	7,200 m^3 @ \$3.0	\$ 21,600
2)	Tamped Impervious Fill	2,100 m^3 @ \$5.0	\$ 10,500
3)	Riprap	100 m^3 @ \$50.0	\$ 10,000
4)	Coarse Filter	860 m^3 @ \$15.00	\$ 12,900
5)	Fine Filter	620 m^3 @ \$18.00	\$ 11,160
6)	Lean Concrete	65 m^3 @ \$200	\$ 13,000
7)	Reinforced Concrete	450 m^3 @ \$400	\$180,000
8)	Miscellaneous		\$ 25,000
			\$284,160
		x 1.15	\$326,784
		Say	\$327,000

F) **Dam Embankment**

1)	Mobilization & Demobilization	L.S.	$ 400,000
2)	Care of Water	L.S.	$ 50,000
3)	Common Excavation	1,248,300 m^3 @ $1.60	$1,997,280
4)	Compacted Impervious Fill	1,021,100 m^3 @ $1.00	$1,021,100
5)	Coarse Filter	94,200 m^3 @ $15.00	$1,413,000
6)	Fine Filter	90,800 m^3 @ $18.00	$1,634,400
7)	Pitrun Gravel	13,400 m^3 @ $8.00	$ 107,200
8)	Riprap	13,350 m^3 @ $50.00	$ 667,500
9)	Road Gravel	550 m^3 @ $10.00	$ 5,500
10)	Topsoil & Seeding	6.4 Ha @ $7000.00	$ 44,480
11)	Wick Drain	L.S.	$ 232,000
12)	Instrumentation	L.S.	$ 35,000
13)	Road Guardrail	L.S.	$ 50,000
14)	Miscellaneous		$ 100,000
			$7,757,780
		x 1.15	$8,921,447
		Say	$8,922,000
			==========

G) Outlet Works

1)	Common Excavation	78,100 m^3 @ $1.6	$ 124,960
2)	Structural Excavation	10,800 m^3 @ $3.0	$ 32,400
3)	Tamped Impervious Backfill	7,800 m^3 @ $5.00	$ 39,000
4)	Riprap	200 m^3 @ $50.00	$ 10,000
5)	Coarse Filter	200 m^3 @ $15.00	$ 3,000
6)	Fine Filter	150 m^3 @ $18.00	$ 2,700
7)	Lean Concrete	45 m^3 @ $200.0	$ 9,000
8)	Reinforced Concrete	2,000 m^3 @ $350	$ 700,000
9)	Bituminous Paint	L.S.	$ 30,000
10)	Gate & Lift - 60" x 84" Sluice Gate	1 Unit @ $50,000	$ 50,000
11)	Air Vent	L.S.	$ 20,000
12)	Safety Ladder	L.S.	$ 60,000
13)	Metal Building	L.S.	$ 100,000
14)	Miscellaneous		$ 50,000
			$1,231,060
		x 1.15	$1,415,719
		Say	$1,420,000

H) Saddle Dyke

1)	Common Excavation	651,000 m^3 @ $1.60	$1,041,600
2)	Compacted Impervious Fill	473,000 m^3 @ $1.00	$ 473,000
3)	Riprap	29,000 m^3 @ $50.00	$1,450,000
4)	Pitrun Gravel	37,000 m^3 @ $8.00	$ 296,000
5)	Coarse Filter	19,000 m^3 @ $15.00	$ 285,000
6)	Fine Filter	9,000 m^3 @ $18.00	$ 162,000
7)	Road Gravel	3,400 m^3 @ $10.0	$ 34,000
8)	Topsoil & Seeding	3.5 Ha @ $7,000	$ 24,500
9)	Road Guardrail	L.S.	$ 15,000
10)	Miscellaneous		$ 50,000
			$3,831,100
		x 1.15	$4,405,765
		Say	$4,410,000

I) Causeway @ Highway # 527

1)	Mobilization & Demobilization	L.S.	\$ 100,000
2)	Traffic Detour & Control	L.S.	\$ 50,000
3)	Common Excavation	334,000 m^3 @ \$1.60	\$ 534,400
4)	Compacted Impervious Fill	261,000 m^3 @ \$1.00	\$ 261,000
5)	Road Subbase & Pavement	L.S.	\$ 42,000
6)	Riprap	19,000 m^3 @ \$50.00	\$ 950,000
7)	Coarse Filter	29,000 m^3 @ \$15.00	\$ 435,000
8)	Fine Filter	23,500 m^3 @ \$18.00	\$ 423,000
9)	Pitrun Gravel	10,600 m^3 @ \$8.00	\$ 84,800
10)	Wick Drain	L.S.	\$ 100,000
11)	SPCSP Extended & Up-graded	L.S.	\$ 50,000
12)	Road Guardrail	L.S.	\$ 30,000
13)	Miscellaneous		\$ 50,000
			\$3,110,200
		x 1.15	\$3,576,730
		Say	\$3,580,000

J) Causeway @ EW Road - North Boundary of 33-14-28-W4

1)	Traffic Detour & Control	L.S.	\$ 20,000
2)	Common Excavation	37,500 m^3 @ \$1.60	\$ 60,000
3)	Compacted Impervious Fill	27,500 m^3 @ \$1.00	\$ 27,500
4)	Riprap	3,730 m^3 @ \$50.0	\$ 186,500
5)	Coarse Filter	2,850 m^3 @ \$15.00	\$ 42,750
6)	Fine Filter	1,700 m^3 @ \$18.00	\$ 30,600
7)	Road Surfacing	L.S.	\$ 30,000
8)	Gated CSP Structure	L.S.	\$ 60,000
9)	Road Guardrail	L.S.	\$ 20,000
10)	Miscellaneous		\$ 20,000
			\$ 497,350
		x 1.15	\$ 571,953
		Say	\$ 572,000

K) Road Relocation

1)	Traffic Detour & Control	L.S.	$ 50,000
2)	Upgraded Gravel Road	3.3. km @ $40,000	$ 132,000
3)	New Gravel Road	4.8 km @ $60,000	$ 288,000
4)	Fencing	9,600 m @ $3.0	$ 28,000
5)	Traffic Sign	L.S.	$ 10,000
6)	Miscellaneous		$ 50,000
			$ 558,800
		x 1.15	$ 642,620
		Say	$ 700,000

L) Reservoir

1)	Reservoir Clearing	600 Ha @ $1,000	$ 600,000
2)	Rim Stabilization	L.S.	$ 200,000
3)	Cultivated Dry Land	140 Ac @ $1,000	$ 140,000
4)	Range Pasture	700 Ac @ $500	$ 350,000
5)	Valley Walls	140 Ac @ $300	$ 42,000
6)	Large Farmstead	1 Unit @ $400,000	$ 400,000
7)	Medium Farmstead	1 Unit @ $200,000	$ 200,000
8)	Small Farmstead	3 Units @ $50,000	$ 150,000
9)	Wells	L.S.	$ 30,000
10)	Refencing	L.S.	$ 50,000
11)	Miscellaneous		$ 50,000
			$2,212,000
		x 1.15	$2,543,800
		Say	$2,700,000

References

Geiger, K. W., (1965); Bedrock Topography of Southwestern Alberta; Research Council of Alberta; Preliminary Report 65-1.

Geiger, K. W., (1968); Bedrock Topography of the Gleichen Map - Area, Alberta; Research Council of Alberta; Report 67-2.

Green, R., (1972); Geological Map of Alberta (1:1,267,000); Alberta Research Council.

Harris, Stuart A. and Waters, R. R., (1977); Late Quaternary History of Southwest Alberta; Bulletin of Canada Petroleum Geology; Volume 25, No. 1, Pages 35-62.

Irish, E. J. W. (1971); Geology - Southern Plains of Alberta (1:500,000); Geological Survey of Canada; Map 1286A.

Ozoray, G. F. and Lytviak, A. T., (1974); Hydrogeology of the Gleichen Area, Alberta; Alberta Research Council; Report 74-9.

Piteau Engineering Ltd., (1985); Hydrogeological Assessment of the Proposed Pine Coulee Reservoir.

Legend for Drawing 1

1. Recent

   C - colluvium consisting of intermixed clay, silt, sand and reworked till and bedrock material.

   AL - floodplain alluvium consisting of interbedded clay, silt, sand and minor amounts of gravel.

2. Glacial

   GF - glaciofluvial deposits consisting of interbedded clay, silt, sand and/or gravel.

   OW - glaciofluvial outwash deposits consisting primarily of sand and gravel with minor amounts of interbedded silt and clay.

   TM - glacial till moraine consisting of silty to clayey till with lenses of sand or silt.

   WM - water modified till moraine consisting of interlayered clay, silt, sand or till with a silty to sandy texture.

3. Bedrock

   B - Areas of exposed or thinly covered bedrock consisting of grey calcareous sandstone interbedded with grey calcareous mudstone or shale of the Porcupine Hills and/or Willow Creek Formations.

4. Geological Boundaries:

   Definite ——

   Inferred – – – –

   Meltwater Channels:

   - Definite ┬┬┬┬ / ┴┴┴┴
   - Approximate ┬ ┬ ┬ ┬ / ┴ ┴ ┴ ┴

   Glacially streamlined features: flute, drumlin or drumlinoid ridges —•—

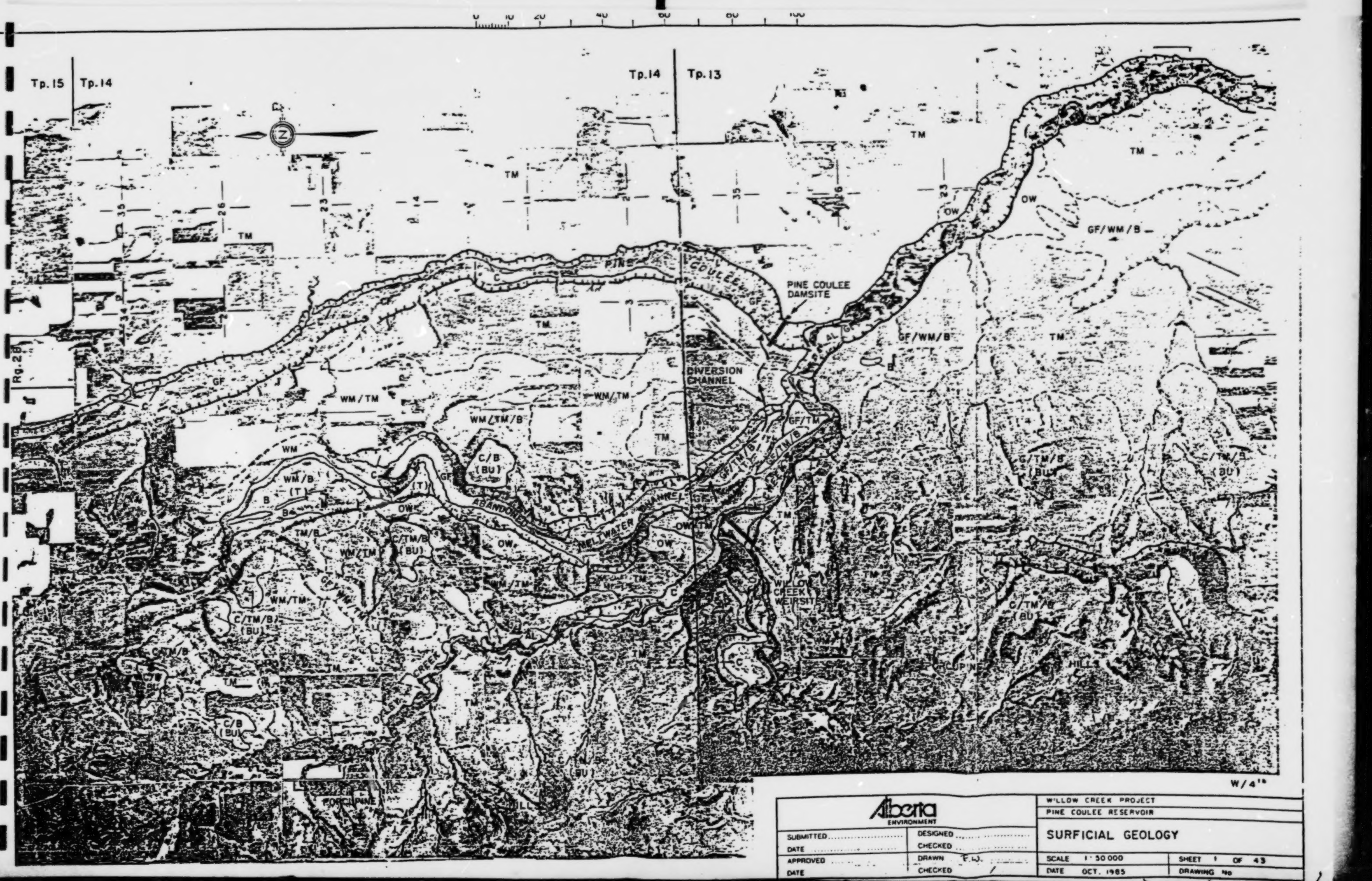
NOTES:

1. Multiple designations (e.g. GF/WM) indicates that both surficial deposits are present with the first unit usually overlying the second.
2. Genetic modifiers are enclosed in brackets.

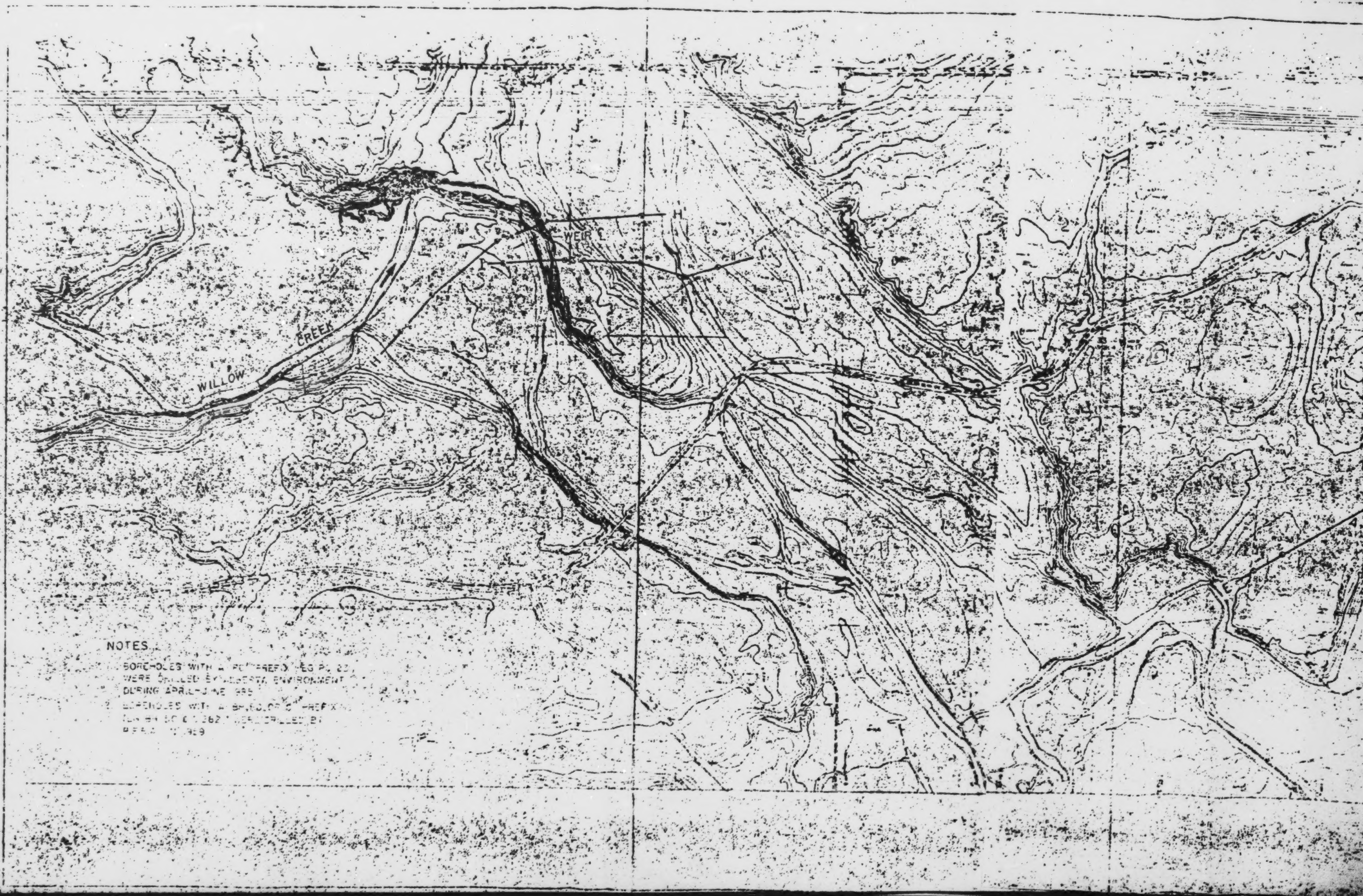
   SM - slope movement

   T - terrace

   BU - bedrock upland area

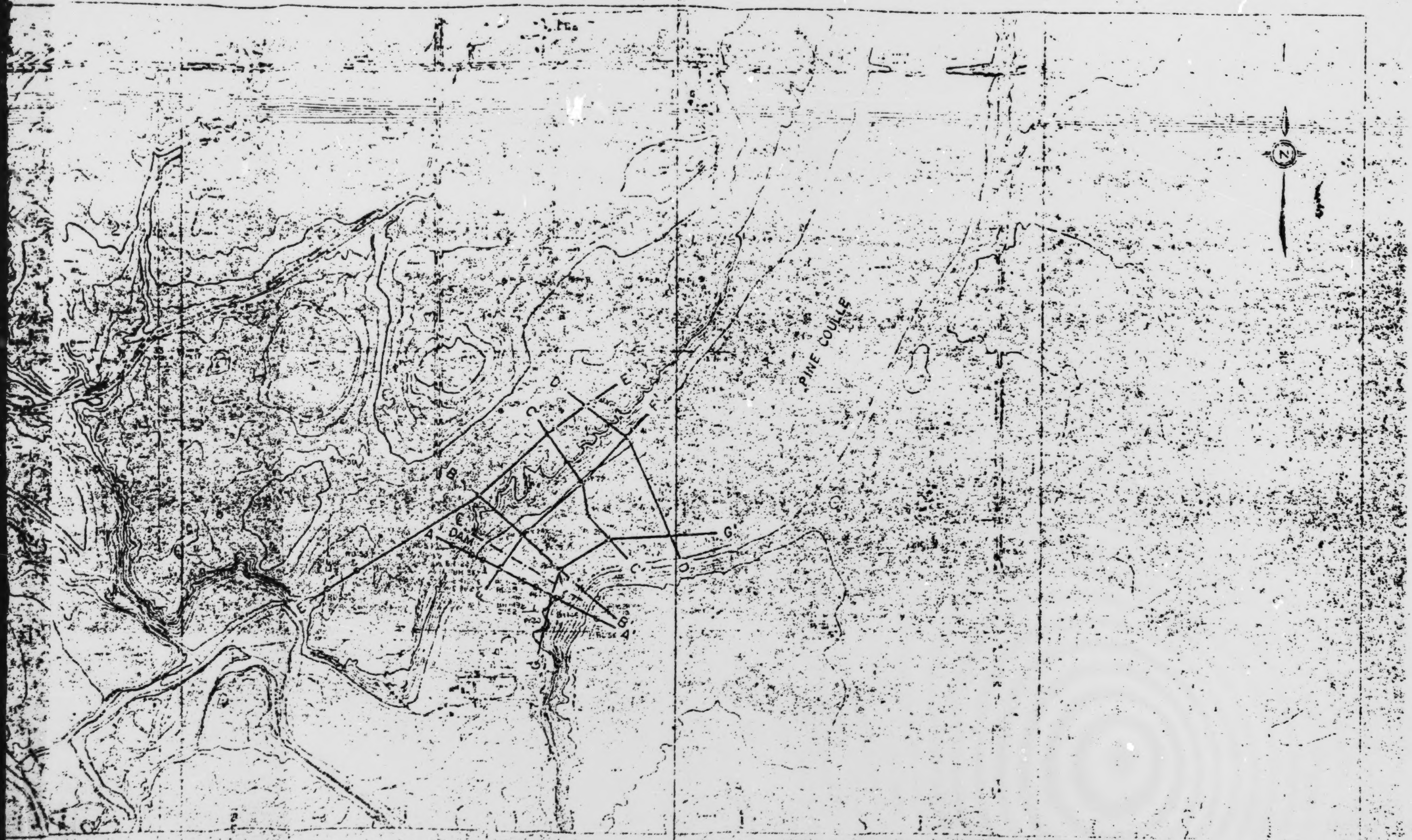

0
10
20
40
60
80
100
Tp. 15
Tp. 14
Tp. 14
Tp. 13
Rg. 28
N
TM
OW
GF/WM/B
PINE COULEE
PINE COULEE DAMSITE
DIVERSION CHANNEL
AL
GF
C
WM/TM
WM/TM/B
GF/TM
C/B (BU)
WM
WM/B (T)
B
(T)
ABANDONED MELTWATER CHANNEL
TM/B
C/TM/B (BU)
G/TM/B (BU)
C/TM/B (BU)
WILLOW CREEK WEIRSITE
WILLOW CREEK
PORCUPINE HILLS
W/4th

Alberta ENVIRONMENT		WILLOW CREEK PROJECT
		PINE COULEE RESERVOIR
SUBMITTED	DESIGNED	SURFICIAL GEOLOGY
DATE	CHECKED	
APPROVED	DRAWN F.W.	SCALE 1 : 50 000 — SHEET 1 OF 43
DATE	CHECKED	DATE OCT. 1985 — DRAWING No


WILLOW CREEK
WEIR
H
H
E

NOTES

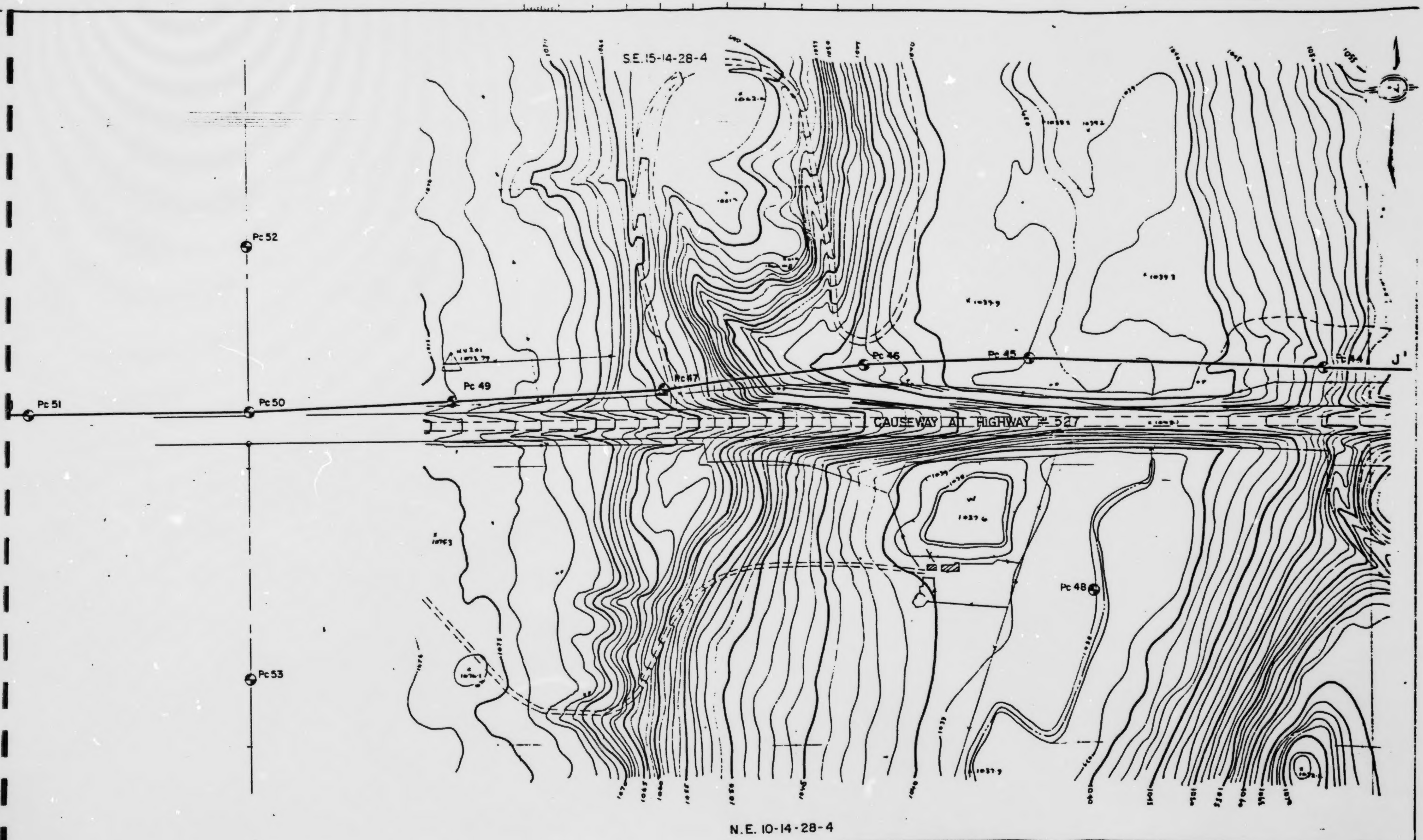
1. BOREHOLES WITH A "PC" PREFIX (EG PC 22) WERE DRILLED BY ALBERTA ENVIRONMENT DURING APRIL-JUNE 1985
2. BOREHOLES WITH A BH, [illegible] "C" PREFIX [illegible] BH SC 6 1362 [illegible] DRILLED BY [illegible] 1989



PINE COULEE
DAM
A
A'
B
B'
C
C'
D
D'
E
E'
F
G'
N

Alberta
ENVIRONMENT

BOREHOLE AND STRATIGRAPHIC SECTIONS
LOCATION PLAN


S.E.15-14-28-4
Pc 52
Pc 51
Pc 50
Pc 49
Pc 47
Pc 46
Pc 45
Pc 44
J'
CAUSEWAY AT HIGHWAY # 527
Pc 48
Pc 53
N.E. 10-14-28-4

Alberta
ENVIRONMENT

SUBMITTED	DESIGNED
DATE	CHECKED
APPROVED	DRAWN
DATE	CHECKED

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

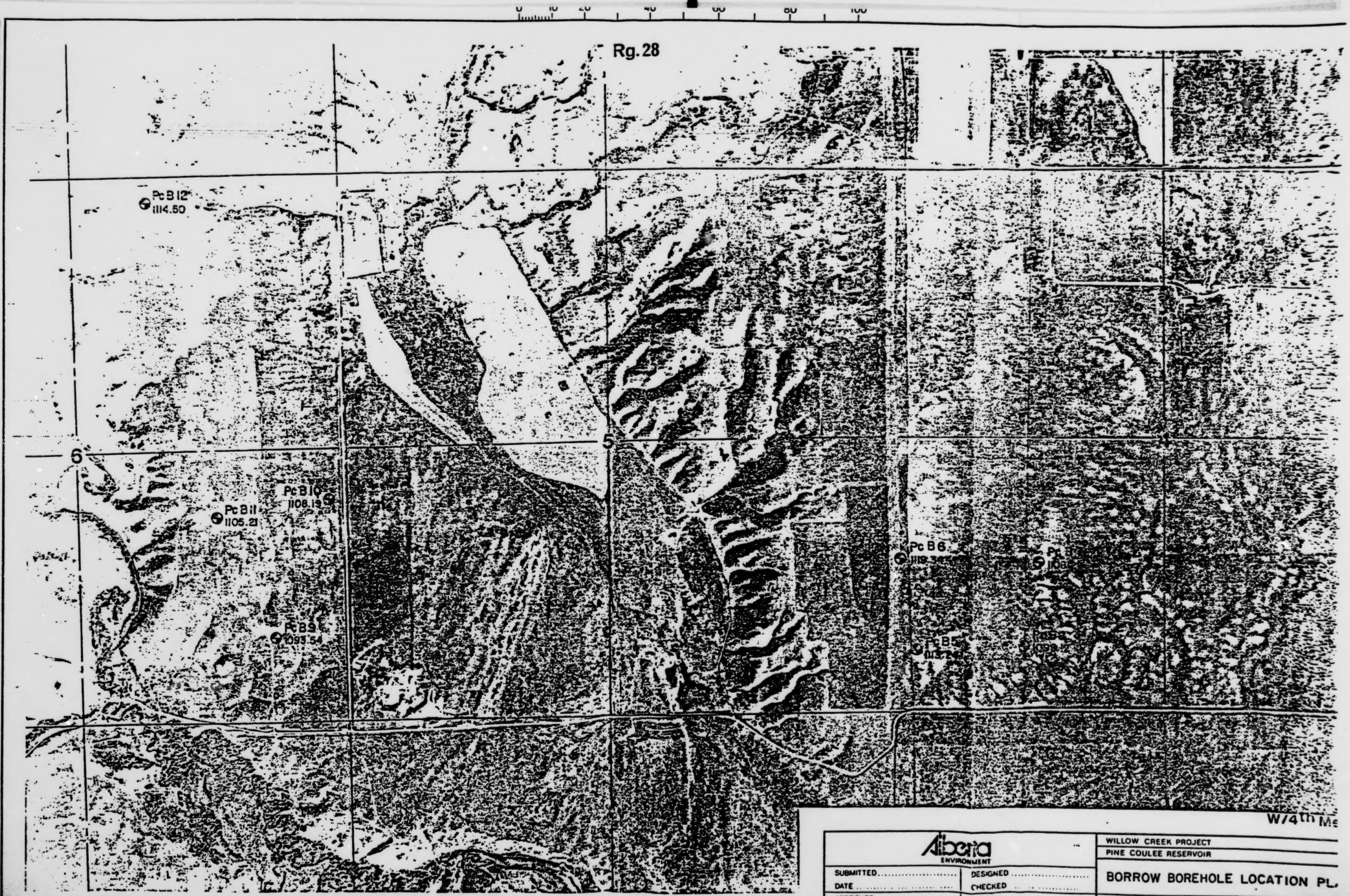
BOREHOLE & STRATIGRAPHIC SECTION J-J
LOCATION PLAN

SCALE 1:2500
SHEET 3 OF 43
DATE OCT. 1985

MICROFILM DATE

DRAWING FILE No PICOR 3785

CENTRAL FILE No


0 10 20 40 60 80 100
Rg. 28
6
5
PcB12 1114.50
PcB11 1105.21
PcB10 1108.19
PcB9 1093.54
PcB6 1118.34
Pc
PcB5
W/4th M

Alberta ENVIRONMENT

SUBMITTED	DESIGNED
DATE	CHECKED
APPROVED	DRAWN
DATE	CHECKED

WILLOW CREEK PROJECT

PINE COULEE RESERVOIR

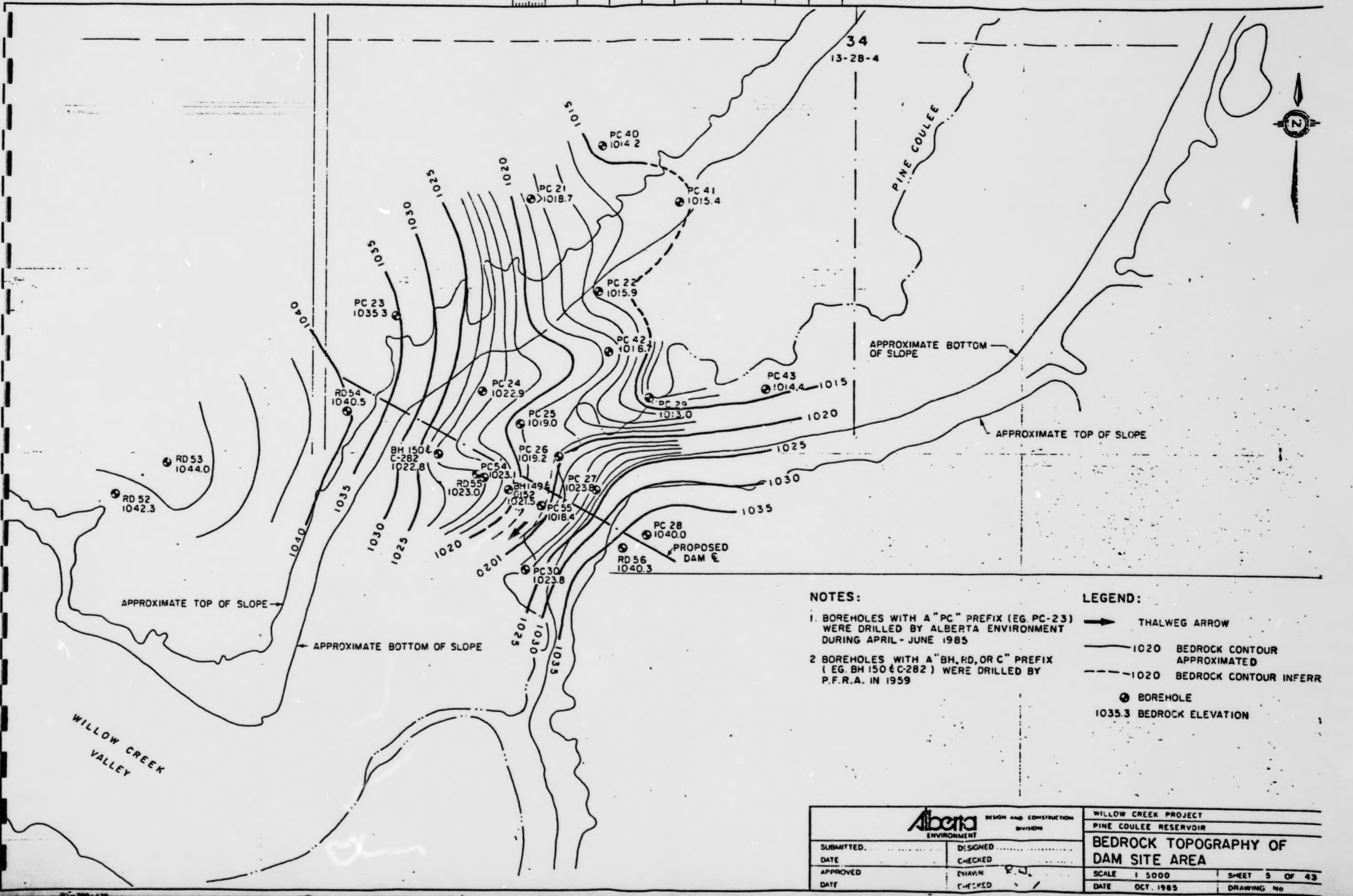
BORROW BOREHOLE LOCATION PL

SCALE 1 10 000

SHEET 4 OF 4

DATE Sept. 1985

DRAWING No PICOR


34
13-28-4
PINE COULEE
PC 40
1014.2
PC 21
1018.7
PC 41
1015.4
PC 22
1015.9
PC 23
1035.3
PC 42
1016.7
PC 24
1022.9
PC 43
1014.4
PC 29
1013.0
PC 25
1019.0
PC 26
1019.2
RD 54
1040.5
RD 53
1044.0
RD 52
1042.3
BH 150 & C-282
1022.8
PC54
1023.1
RD55
1023.0
BH149 & C152
1021.5
PC 27
1023.8
PC 55
1018.4
PC 28
1040.0
RD 56
1040.3
PC30
1023.8
PROPOSED DAM ℄
APPROXIMATE BOTTOM OF SLOPE
APPROXIMATE TOP OF SLOPE
APPROXIMATE TOP OF SLOPE
APPROXIMATE BOTTOM OF SLOPE
WILLOW CREEK VALLEY
1015
1020
1025
1030
1035
1040
N

NOTES:

1. BOREHOLES WITH A "PC" PREFIX (EG. PC-23) WERE DRILLED BY ALBERTA ENVIRONMENT DURING APRIL - JUNE 1985
2. BOREHOLES WITH A "BH, RD, OR C" PREFIX (EG. BH 150 & C-282) WERE DRILLED BY P.F.R.A. IN 1959

LEGEND:

➡	THALWEG ARROW
——1020	BEDROCK CONTOUR APPROXIMATED
- - - -1020	BEDROCK CONTOUR INFERR
●	BOREHOLE
1035.3	BEDROCK ELEVATION

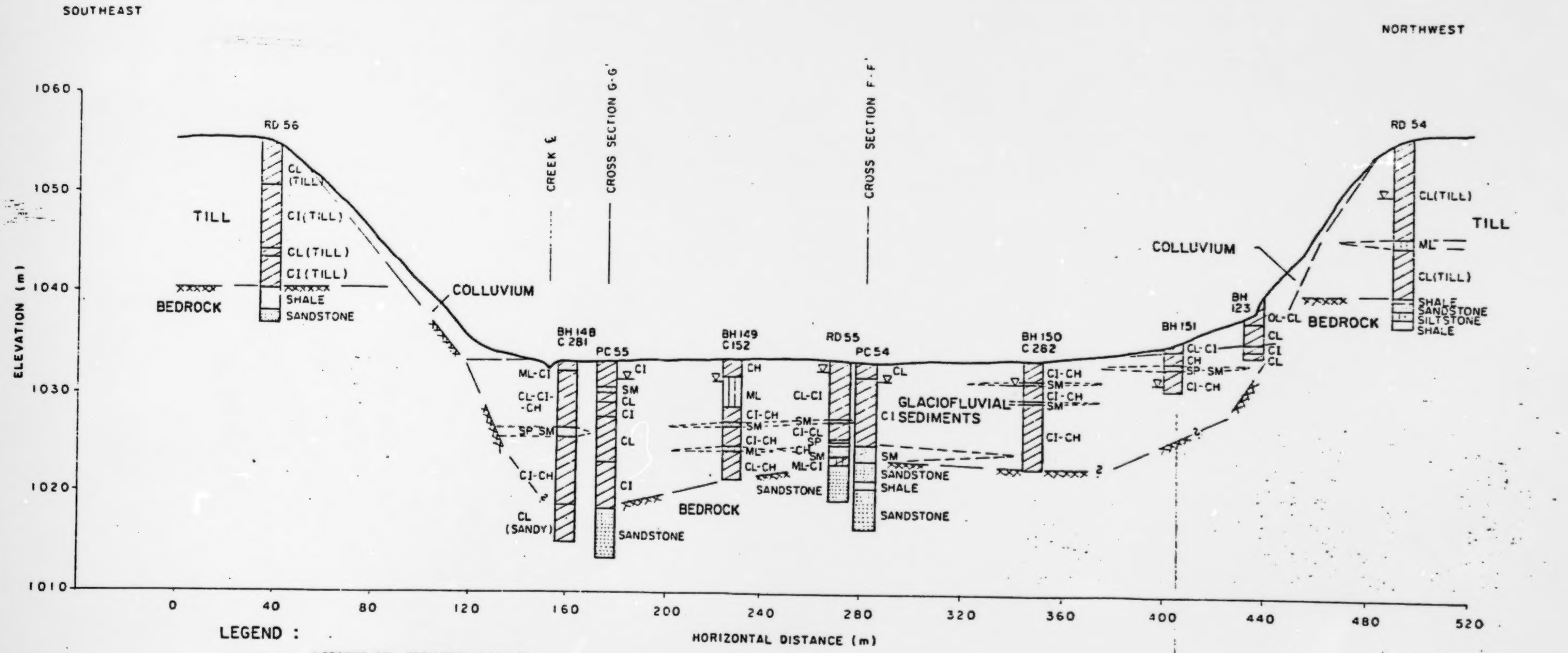
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

BEDROCK TOPOGRAPHY OF DAM SITE AREA

SUBMITTED.		DESIGNED	
DATE		CHECKED	
APPROVED		DRAWN	R.W.
DATE		CHECKED	

SCALE 1 5000
SHEET 5 OF 43
DATE OCT. 1985
DRAWING No


SOUTHEAST
NORTHWEST
ELEVATION (m)
1060
1050
1040
1030
1020
1010
0
40
80
120
160
200
240
280
320
360
400
440
480
520
HORIZONTAL DISTANCE (m)
RD 56
CL (TILL)
CI(TILL)
CL(TILL)
CI(TILL)
SHALE
SANDSTONE
TILL
BEDROCK
COLLUVIUM
CREEK ℄
CROSS SECTION G-G'
CROSS SECTION F-F'
BH 148 C 281
ML-CI
CL-CI--CH
SP-SM
CI-CH
CL (SANDY)
PC 55
CI
SM
CL
CI
CL
CI
SANDSTONE
BEDROCK
BH 149 C 152
CH
ML
CI-CH
SM
CI-CH
ML
CL-CH
SANDSTONE
RD 55
CL-CI
SM
CI-CL
SP
CH
SM
ML-CI
PC 54
CL
CI
SM
SANDSTONE
SHALE
SANDSTONE
GLACIOFLUVIAL SEDIMENTS
BH 150 C 282
CI-CH
SM
CI-CH
SM
CI-CH
BH 151
CL-CI
CH
SP-SM
CI-CH
BH 123
OL-CL
CL
CI
CL
BEDROCK
COLLUVIUM
RD 54
CL(TILL)
TILL
ML
CL(TILL)
SHALE
SANDSTONE
SILTSTONE
SHALE

LEGEND :

- —— —— INFERRED SOIL FORMATION CONTACT
- - - - - - - INFERRED SOIL TYPE CONTACT
- xxxxx xxxx INFERRED SOIL BEDROCK CONTACT
- ∇ WATER LEVEL IN BOREHOLE TAKEN IMMEDIATELY AFTER DRILLING
- ○ FLOWING WELL
- ← SLICKENSIDES

SOIL SYMBOLS

- ▨ CLAY (CL, CI, CH)
- ☐ SILT (ML)
- ☐ SAND (SM, SP, SC)
- ▩ GRAVEL (GM, GP, GW)

BEDROCK SYMBOLS

- ☐ SHALE
- ☐ SANDSTONE

STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED
DATE
APPROVED
DATE

DESIGNED
CHECKED
DRAWN R.H.
CHECKED /

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

STRATIGRAPHIC SECTION A-A' ~40m D/S OF PINE COULEE DAM SITE ℄

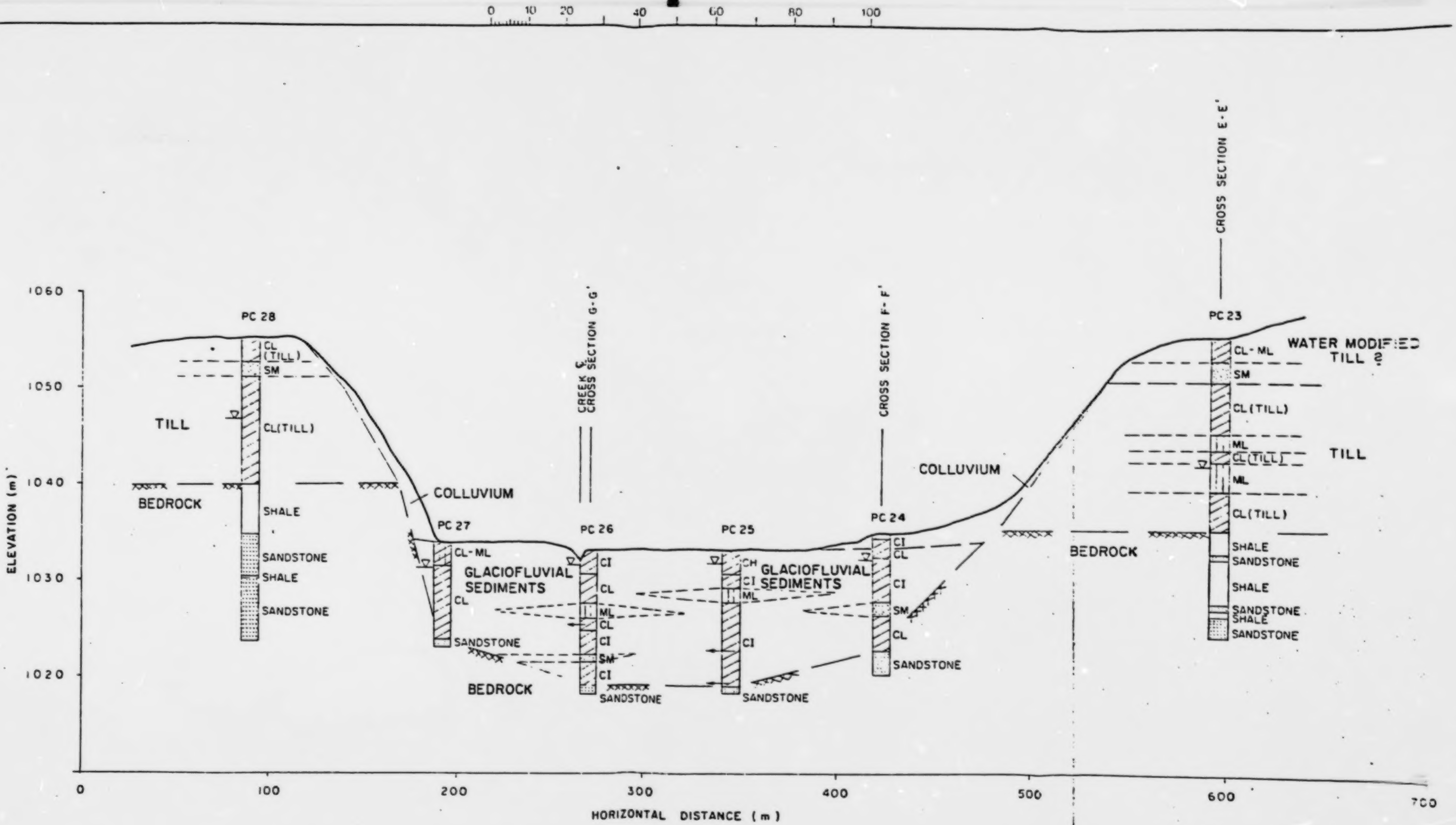
SCALE AS SHOWN
SHEET 6 OF 43
DATE OCT 1985
DRAWING No

MICP
DATE
DRAWING FILE No
CENTRAL FILE No
PJ-200•4.10

MICROFILM DATE

DRAWING FILE No

CENTRAL FILE No


0 10 20 40 60 80 100
ELEVATION (m)
1060
1050
1040
1030
1020
HORIZONTAL DISTANCE (m)
0
100
200
300
400
500
600
700
PC 28
CL (TILL)
SM
CL(TILL)
SHALE
SANDSTONE
SHALE
SANDSTONE
TILL
BEDROCK
COLLUVIUM
PC 27
CL-ML
CL
SANDSTONE
GLACIOFLUVIAL SEDIMENTS
BEDROCK
CREEK ℄ CROSS SECTION G-G'
PC 26
CI
CL
ML
CL
CI
SM
CI
SANDSTONE
PC 25
CH
CI
ML
CI
SANDSTONE
GLACIOFLUVIAL SEDIMENTS
CROSS SECTION F-F'
PC 24
CI
CL
CI
SM
CL
SANDSTONE
COLLUVIUM
BEDROCK
CROSS SECTION E-E'
PC 23
CL-ML
SM
CL(TILL)
ML
CL(TILL)
ML
CL(TILL)
SHALE
SANDSTONE
SHALE
SANDSTONE
SHALE
SANDSTONE
WATER MODIFIED TILL ?
TILL

LEGEND:

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

SUBMITTED	DESIGNED
DATE	CHECKED
APPROVED	DRAWN L.W.
DATE	CHECKED

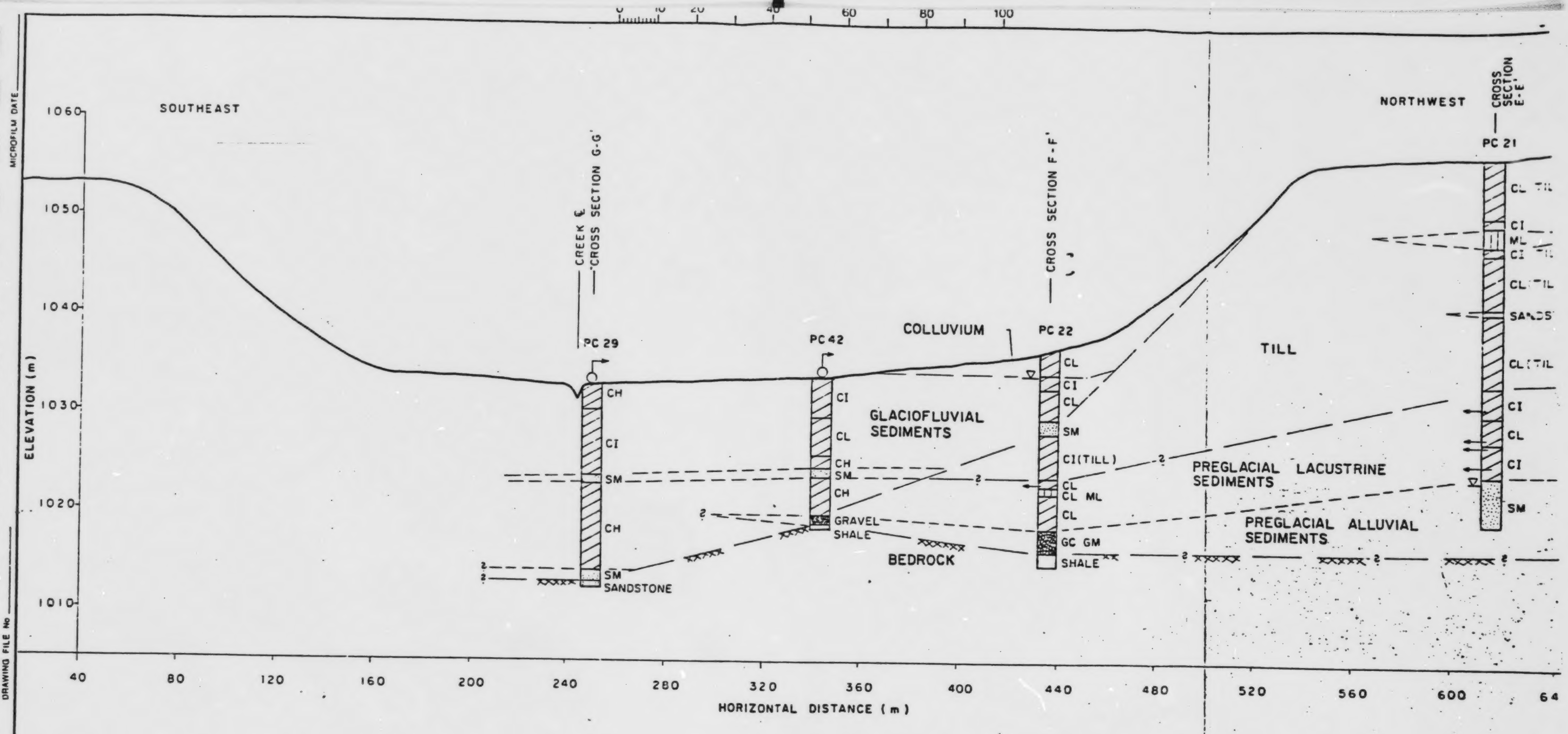
WILLOW CREEK PROJECT

PINE COULEE RESERVOIR

STRATIGRAPHIC SECTION B-B'
~60m U/S OF PINE COULEE DAM SITE ℄

SCALE AS SHOWN SHEET 7 OF 43

DATE OCT. 1985 DRAWING No


0
10
20
40
60
80
100
SOUTHEAST
NORTHWEST
CROSS SECTION E-E'
PC 21
1060
1050
1040
1030
1020
1010
ELEVATION (m)
CREEK ℄
CROSS SECTION G-G'
CROSS SECTION F-F'
PC 29
PC 42
PC 22
COLLUVIUM
TILL
GLACIOFLUVIAL SEDIMENTS
PRECLACIAL LACUSTRINE SEDIMENTS
PREGLACIAL ALLUVIAL SEDIMENTS
BEDROCK
CH
CI
SM
CH
SM
SANDSTONE
CI
CL
CH
SM
CH
GRAVEL
SHALE
CL
CI
CL
SM
CI(TILL)
CL
CL ML
CL
GC GM
SHALE
CL TIL
CI
ML
CI TIL
CL TIL
SANDS
CL TIL
CI
CL
CI
SM
?
40
80
120
160
200
240
280
320
360
400
440
480
520
560
600
64
HORIZONTAL DISTANCE (m)

NOTE :

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

MICROFILM DATE
DRAWING FILE No
CENTRAL FILE No

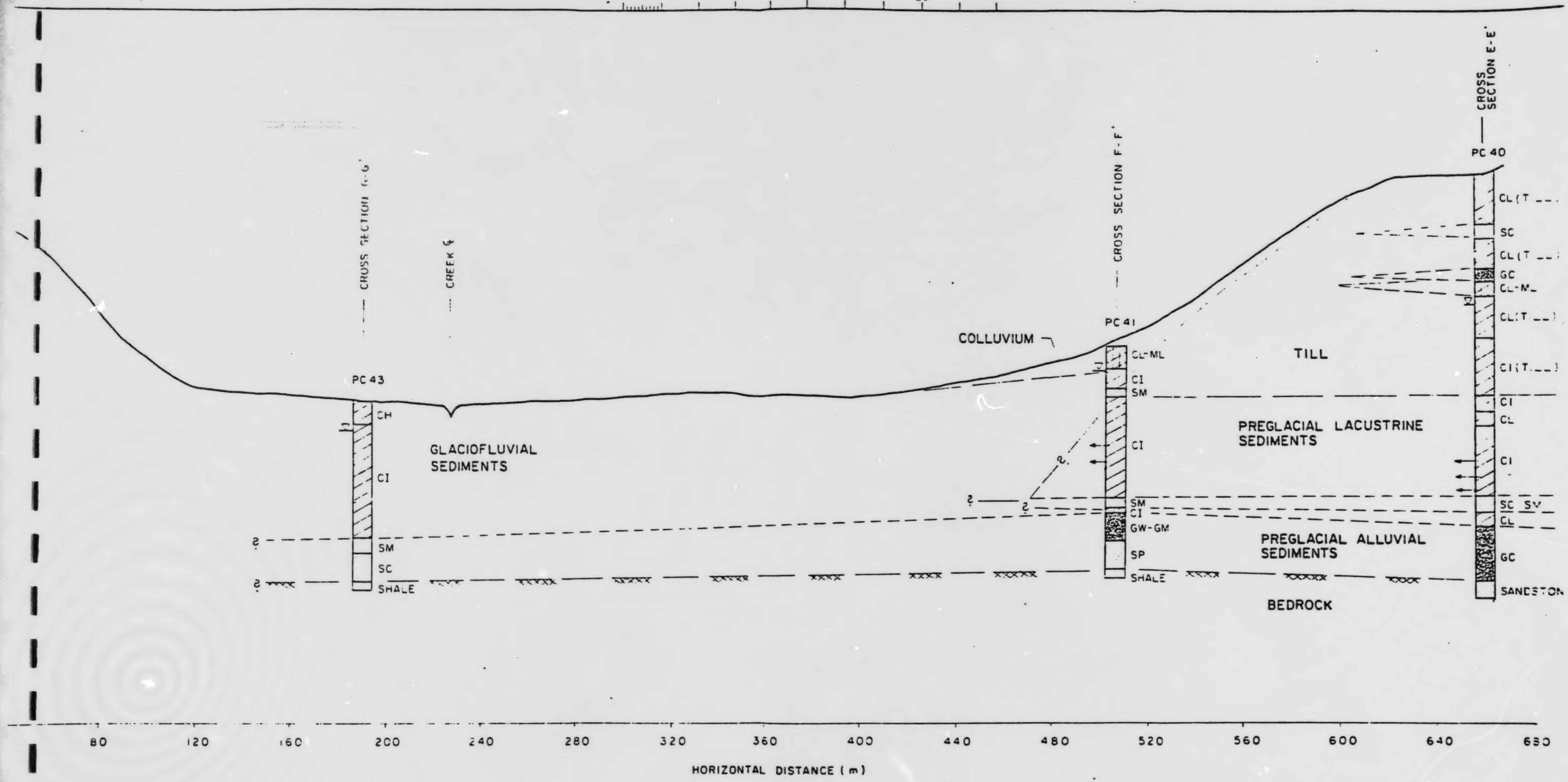
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED	DESIGNED
DATE	CHECKED
APPROVED	DRAWN R.V
DATE	CHECKED

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

STRATIGRAPHIC SECTION C - C' ~250m U/S OF PINE COULEE DAM SITE

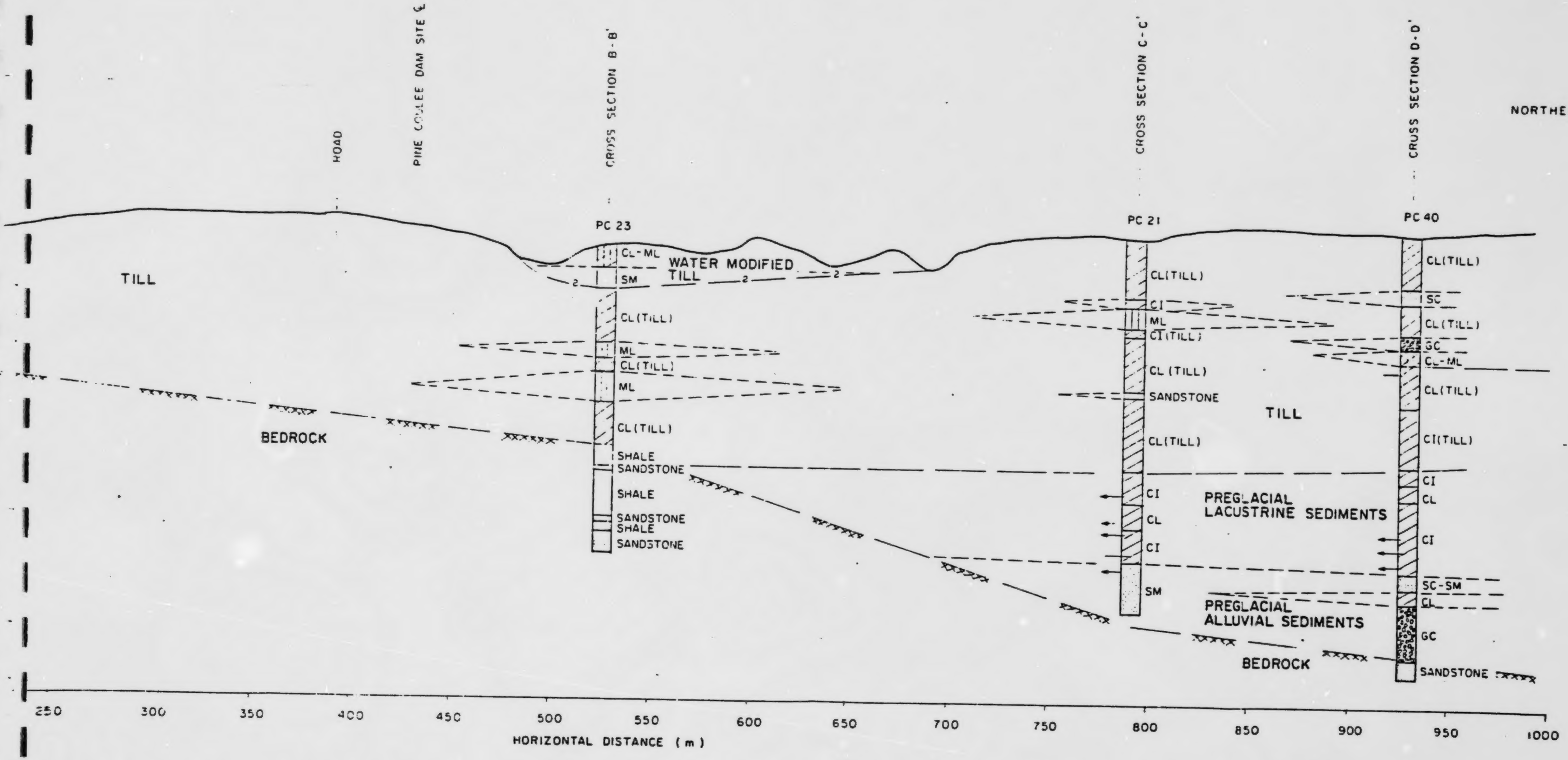
SCALE AS SHOWN SHEET 8 OF 43
DATE OCT 1983 DRAWING No


CROSS SECTION G-G'
CREEK ℄
CROSS SECTION F-F'
CROSS SECTION E-E'
PC 43
PC 41
PC 40
COLLUVIUM
TILL
GLACIOFLUVIAL SEDIMENTS
PREGLACIAL LACUSTRINE SEDIMENTS
PREGLACIAL ALLUVIAL SEDIMENTS
BEDROCK
CH
CI
SM
SC
SHALE
CL-ML
CI
SM
CI
SM
CI
GW-GM
SP
SHALE
CL(T
SC
CL(T
GC
CL-ML
CL(T
CI(T
CI
CL
CI
SC SV
CL
GC
SANDSTON
?
80
120
160
200
240
280
320
360
400
440
480
520
560
600
640
680
HORIZONTAL DISTANCE (m)

LEGEND:

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

Alberta ENVIRONMENT	DESIGN AND CONSTRUCTION DIVISION	WILLOW CREEK PROJECT PINE COULEE RESERVOIR
SUBMITTED DATE APPROVED DATE	DESIGNED CHECKED DRAWN K.W. CHECKED	STRATIGRAPHIC SECTION D - D' ~350m U/S OF PINE COULEE DAM SITE ℄
		SCALE AS SHOWN SHEET 9 OF 43 DATE OCT 1985

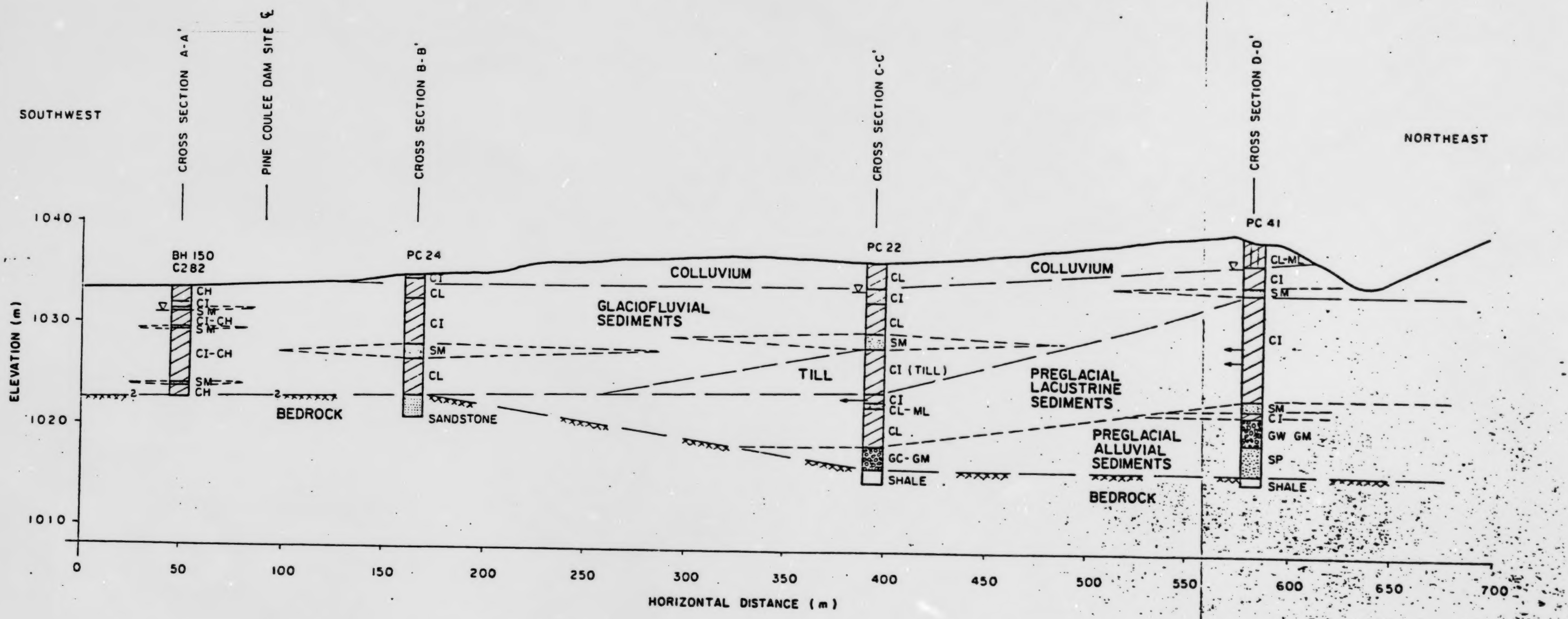

NORTHEAST
ROAD
PINE COULEE DAM SITE ℄
CROSS SECTION B-B'
CROSS SECTION C-C'
CROSS SECTION D-D'
PC 23
PC 21
PC 40
TILL
WATER MODIFIED TILL
2
CL-ML
SM
CL(TILL)
ML
CL(TILL)
ML
CL(TILL)
SHALE
SANDSTONE
SHALE
SANDSTONE
SHALE
SANDSTONE
BEDROCK
CL(TILL)
CI
ML
CI(TILL)
CL (TILL)
SANDSTONE
CL(TILL)
CI
CL
CI
SM
TILL
PREGLACIAL LACUSTRINE SEDIMENTS
PREGLACIAL ALLUVIAL SEDIMENTS
BEDROCK
CL(TILL)
SC
CL(TILL)
GC
CL-ML
CL(TILL)
CI(TILL)
CI
CL
CI
SC-SM
CL
GC
SANDSTONE
250
300
350
400
450
500
550
600
650
700
750
800
850
900
950
1000
HORIZONTAL DISTANCE (m)

LEGEND:

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED	DESIGNED	WILLOW CREEK PROJECT
DATE	CHECKED	PINE COULEE RESERVOIR
APPROVED	DRAWN T.W.	STRATIGRAPHIC SECTION E-E' NORTHWEST (RIGHT) ABUTMENT
DATE		SCALE AS SHOWN SHEET 10 OF 43 DATE OCT 198


SOUTHWEST
NORTHEAST
CROSS SECTION A-A'
PINE COULEE DAM SITE ℄
CROSS SECTION B-B'
CROSS SECTION C-C'
CROSS SECTION D-D'
BH 150
C282
PC 24
PC 22
PC 41
ELEVATION (m)
1040
1030
1020
1010
CH
CI
SM
CI-CH
SM
CI-CH
SM
CH
CI
CL
CI
SM
CL
SANDSTONE
CL
CI
CL
SM
CI (TILL)
CI
CL-ML
CL
GC-GM
SHALE
CL-ML
CI
SM
CI
SM
CI
GW GM
SP
SHALE
COLLUVIUM
COLLUVIUM
GLACIOFLUVIAL SEDIMENTS
TILL
PREGLACIAL LACUSTRINE SEDIMENTS
PREGLACIAL ALLUVIAL SEDIMENTS
BEDROCK
BEDROCK
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
HORIZONTAL DISTANCE (m)

NOTE:

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

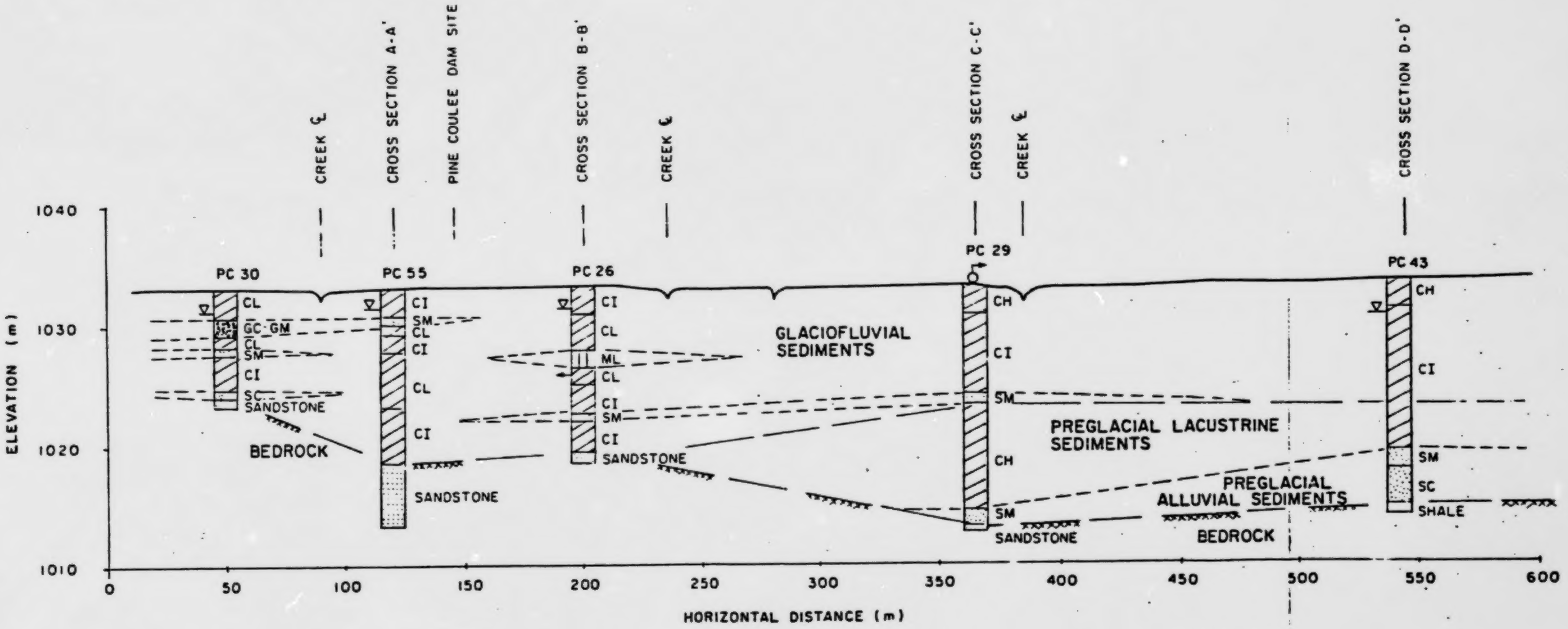
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED		DESIGNED	
DATE		CHECKED	
APPROVED		DRAWN	R.W.
DATE		CHECKED	/

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

STRATIGRAPHIC SECTION F-F'
NORTHWEST SIDE OF VALLEY BOTTO

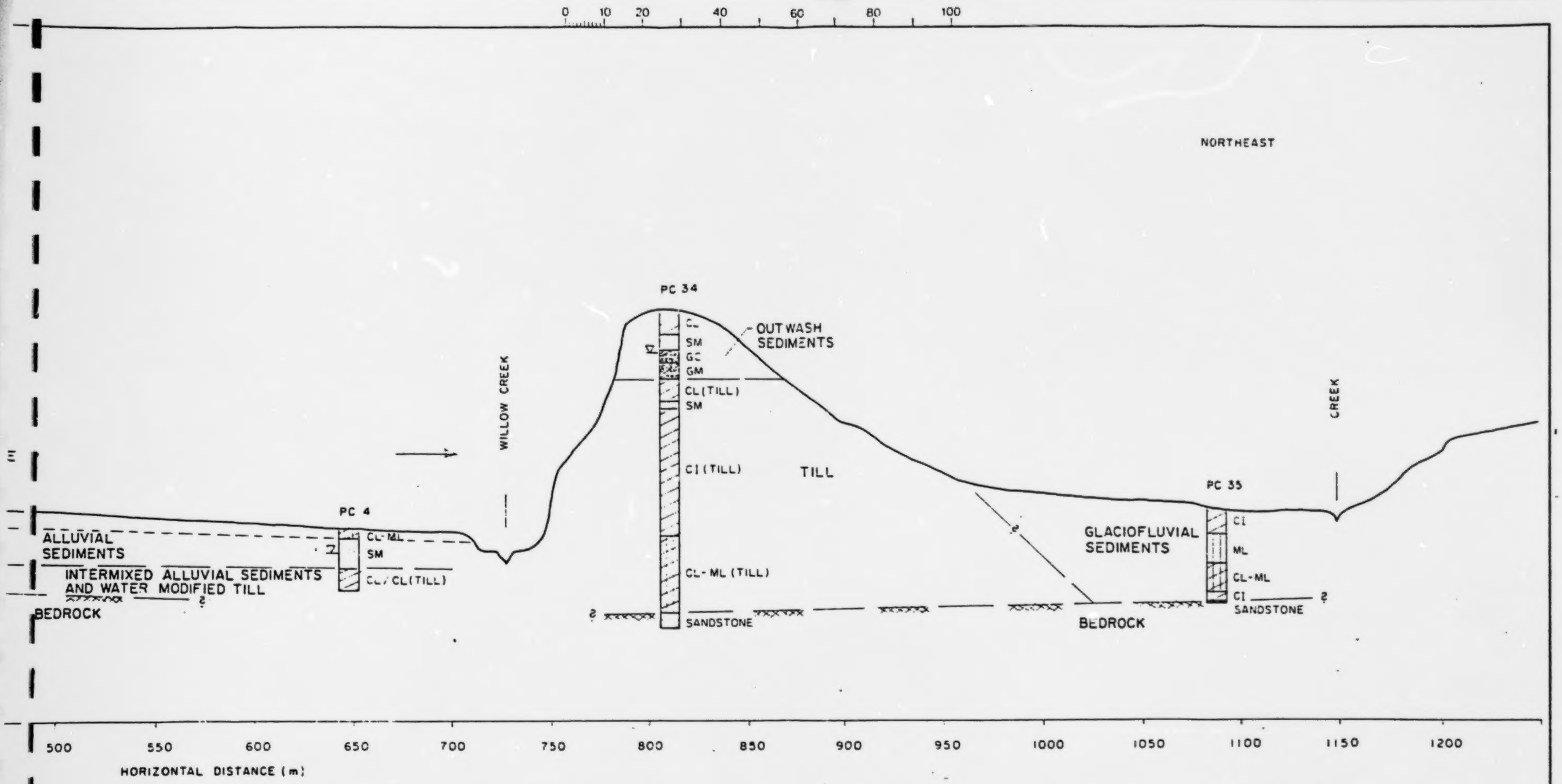
SCALE AS SHOWN	SHEET 11 OF 43
DATE OCT 1985	DRAWING No.


CREEK ℄
CROSS SECTION A-A'
PINE COULEE DAM SITE ℄
CROSS SECTION B-B'
CREEK ℄
CROSS SECTION C-C'
CREEK ℄
CROSS SECTION D-D'
PC 30
PC 55
PC 26
PC 29
PC 43
CL
GC-GM
CL
SM
CI
SC
SANDSTONE
CI
SM
CL
CI
CL
CI
SANDSTONE
CI
CL
ML
CL
CI
SM
CI
SANDSTONE
CH
CI
SM
CH
SM
SANDSTONE
CH
CI
SM
SC
SHALE
BEDROCK
GLACIOFLUVIAL SEDIMENTS
PREGLACIAL LACUSTRINE SEDIMENTS
PREGLACIAL ALLUVIAL SEDIMENTS
BEDROCK
ELEVATION (m)
1040
1030
1020
1010
0
50
100
150
200
250
300
350
400
450
500
550
600
HORIZONTAL DISTANCE (m)

NOTE:

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

ALBERTA ENVIRONMENT	DESIGN AND CONSTRUCTION DIVISION	WILLOW CREEK PROJECT	
		PINE COULEE RESERVOIR	
		STRATIGRAPHIC SECTION G - G' SOUTHEAST SIDE OF VALLEY BOTTC	
SUBMITTED	DESIGNED		
DATE	CHECKED		
APPROVED	DRAWN	SCALE AS SHOWN	SHEET 12 OF 4
DATE		DATE OCT 1985	DRAWING NO



0 10 20 40 60 80 100
NORTHEAST
PC 34
CL
SM
GC
GM
CL(TILL)
SM
CI (TILL)
CL- ML (TILL)
SANDSTONE
OUTWASH SEDIMENTS
TILL
WILLOW CREEK
CREEK
PC 4
CL-ML
SM
CL/CL(TILL)
ALLUVIAL SEDIMENTS
INTERMIXED ALLUVIAL SEDIMENTS AND WATER MODIFIED TILL
BEDROCK
PC 35
CI
ML
CL-ML
CI
SANDSTONE
GLACIOFLUVIAL SEDIMENTS
BEDROCK
?
500
550
600
650
700
750
800
850
900
950
1000
1050
1100
1150
1200
HORIZONTAL DISTANCE (m)

NOTE:

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

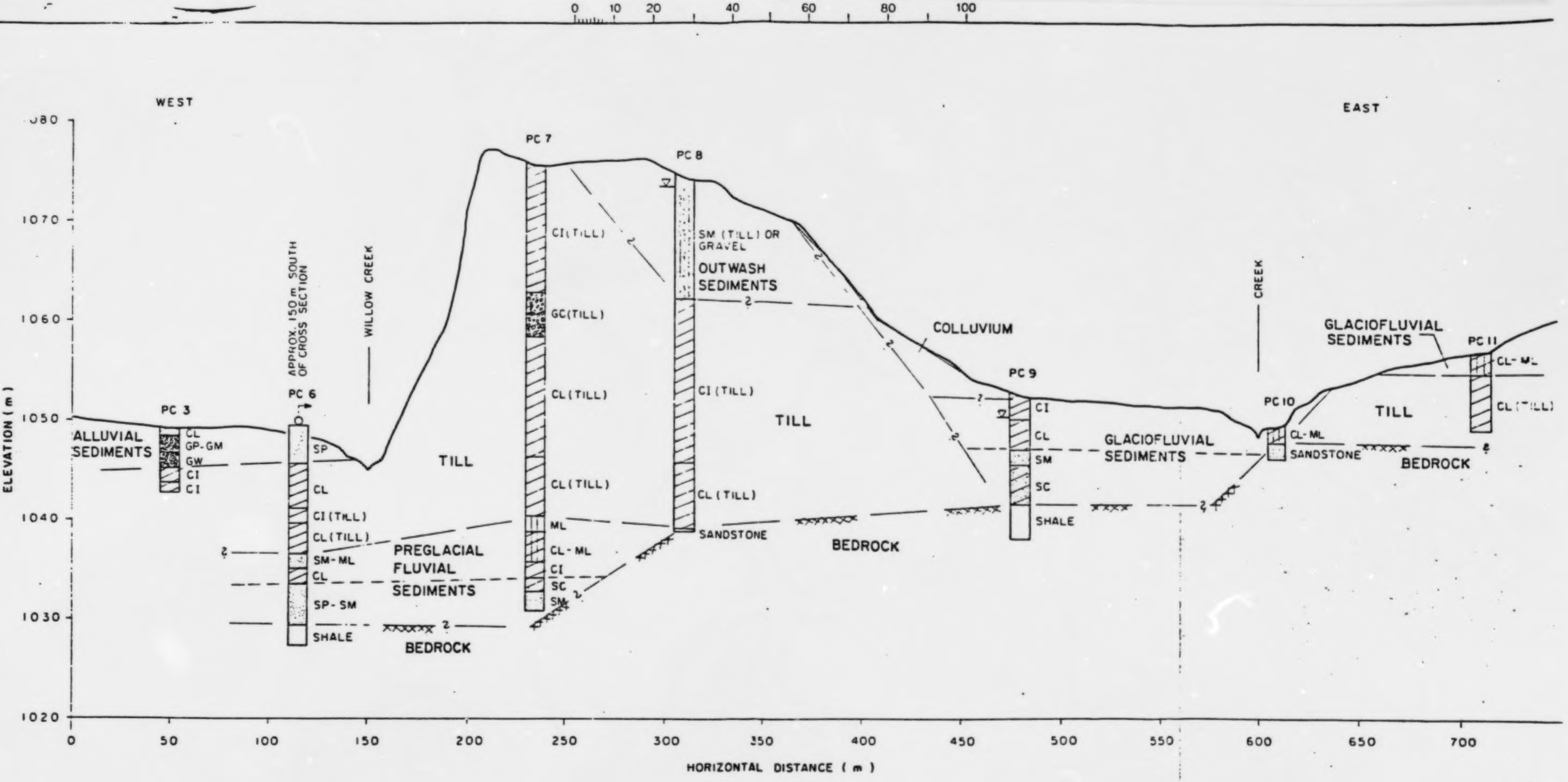
Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

SUBMITTED
DATE
APPROVED
DESIGNED
CHECKED
DRAWN

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

STRATIGRAPHIC SECTION H - H'
~25 m U/S OF WILLOW CREEK WEIR SITE

SCALE AS SHOWN
SHEET 13 OF 43
DATE OCT 1985

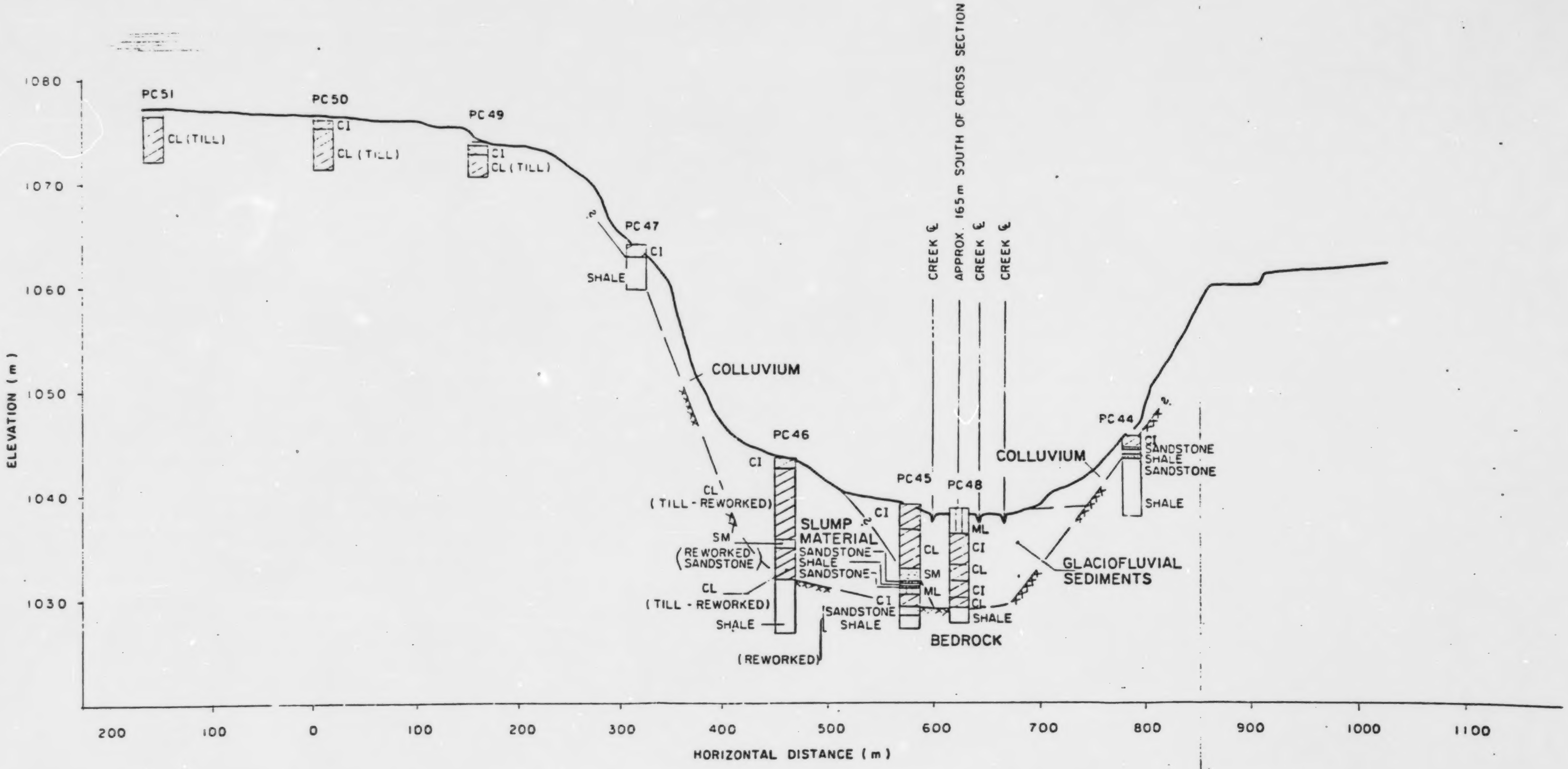

0
10
20
40
60
80
100
WEST
EAST
ELEVATION (m)
1080
1070
1060
1050
1040
1030
1020
HORIZONTAL DISTANCE (m)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
PC 3
PC 6
PC 7
PC 8
PC 9
PC 10
PC 11
APPROX. 150 m SOUTH OF CROSS SECTION
WILLOW CREEK
CREEK
ALLUVIAL SEDIMENTS
CL
GP-GM
GW
CI
CI
SP
CL
CI (TILL)
CL (TILL)
SM-ML
CL
SP-SM
SHALE
PREGLACIAL FLUVIAL SEDIMENTS
TILL
BEDROCK
CI (TILL)
GC (TILL)
CL (TILL)
CL (TILL)
ML
CL-ML
CI
SC
SM
SM (TILL) OR GRAVEL
OUTWASH SEDIMENTS
CI (TILL)
CL (TILL)
SANDSTONE
BEDROCK
TILL
COLLUVIUM
CI
CL
SM
SC
SHALE
GLACIOFLUVIAL SEDIMENTS
CL-ML
SANDSTONE
BEDROCK
TILL
GLACIOFLUVIAL SEDIMENTS
CL-ML
CL (TILL)
?

NOTE :

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

Alberta ENVIRONMENT	DESIGN AND CONSTRUCTION DIVISION	WILLOW CREEK PROJECT PINE COULEE RESERVOIR STRATIGRAPHIC SECTION I-I' ~80m D/S OF WILLOW CREEK WEIR SITE
SUBMITTED	DESIGNED	
DATE	CHECKED	

MICROFILM DATE
DRAWING FILE No


PC51
CL (TILL)
PC50
CI
CL (TILL)
PC49
CI
CL (TILL)
PC47
CI
SHALE
COLLUVIUM
PC46
CI
CL
(TILL - REWORKED)
SM
(REWORKED SANDSTONE)
CL
(TILL - REWORKED)
SHALE
(REWORKED)
SLUMP MATERIAL
SANDSTONE
SHALE
SANDSTONE
SANDSTONE
SHALE
PC45
CI
CL
SM
ML
CI
PC48
ML
CI
CL
CI
CL
SHALE
BEDROCK
CREEK ℄
APPROX. 165m SOUTH OF CROSS SECTION
CREEK ℄
CREEK ℄
COLLUVIUM
GLACIOFLUVIAL SEDIMENTS
PC44
CI
SANDSTONE
SHALE
SANDSTONE
SHALE
ELEVATION (m)
1080
1070
1060
1050
1040
1030
200
100
0
100
200
300
400
500
600
700
800
900
1000
1100
HORIZONTAL DISTANCE (m)

LEGEND:

- FOR LEGEND REFER TO SHEET 6
- STRATIGRAPHIC SECTION LOCATION SHOWN ON SHEET 2

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED
DATE
APPROVED
DATE

DESIGNED
CHECKED
DRAWN R.W.
CHECKED

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

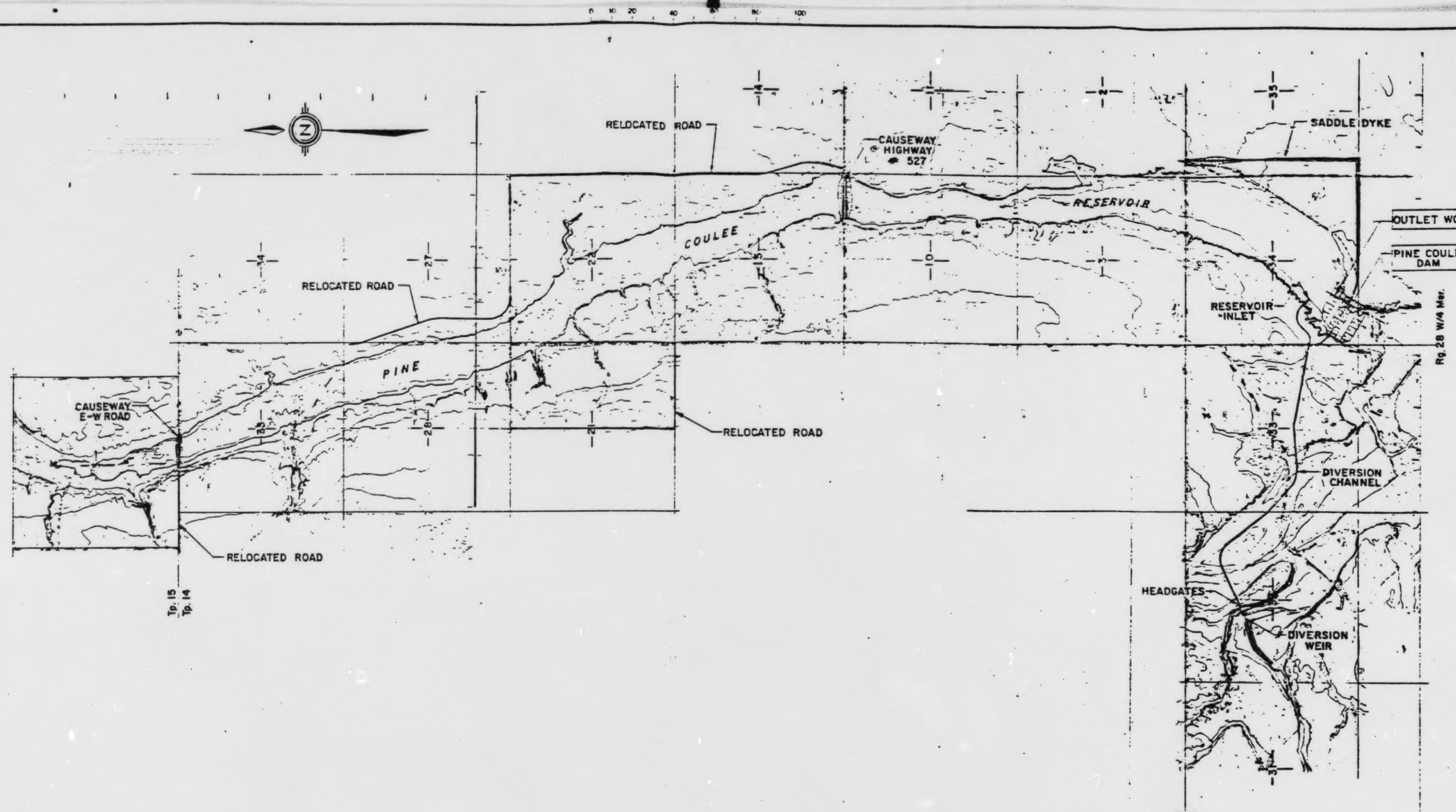
STRATIGRAPHIC SECTION J-J'
40m NORTH OF HWY #527

SCALE AS SHOWN
DATE OCT 1985
SHEET 13 OF 4
DRAWING No

MICROFILM DATE

DRAWING FILE No. PICOR 0785

PRINTAL FILE No.


0 10 20 40 60 80 100
N
RELOCATED ROAD
CAUSEWAY HIGHWAY # 527
SADDLE DYKE
RESERVOIR
OUTLET WORKS
PINE COULEE DAM
COULEE
RELOCATED ROAD
RESERVOIR INLET
Rg. 28 W/4 Mer.
PINE
CAUSEWAY E-W ROAD
RELOCATED ROAD
DIVERSION CHANNEL
RELOCATED ROAD
HEADGATES
DIVERSION WEIR
Tp. 15
Tp. 14
Tp. 14
Tp. 13
14
11
2
35
34
27
22
15
10
3
34
33
28
21
33
3

MARK	NATURE OF REVISION	DATE	ENG	DWN
5				
4				
3				
2				
1				

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

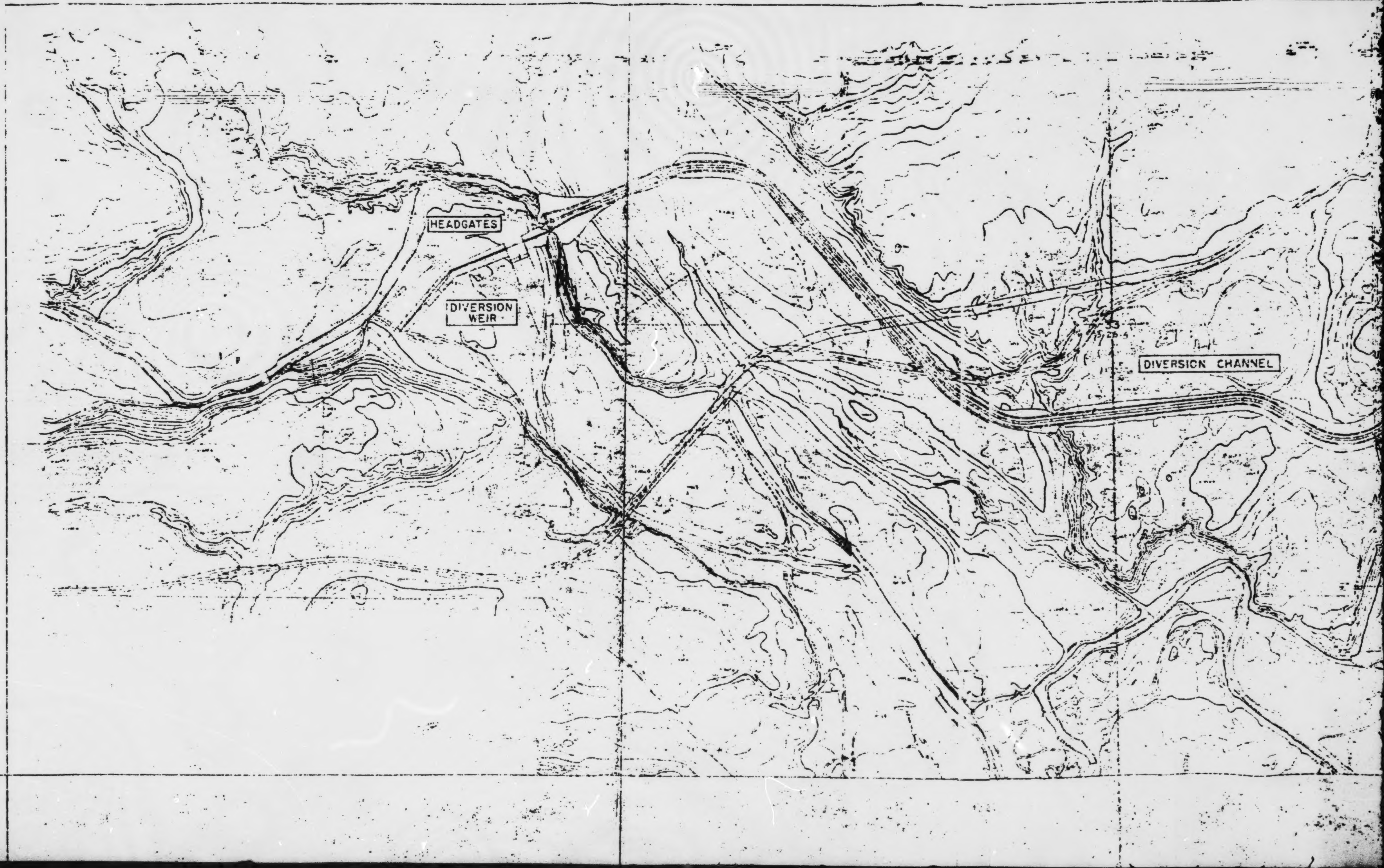
SUBMITTED ....
DATE ....
APPROVED ....
DATE

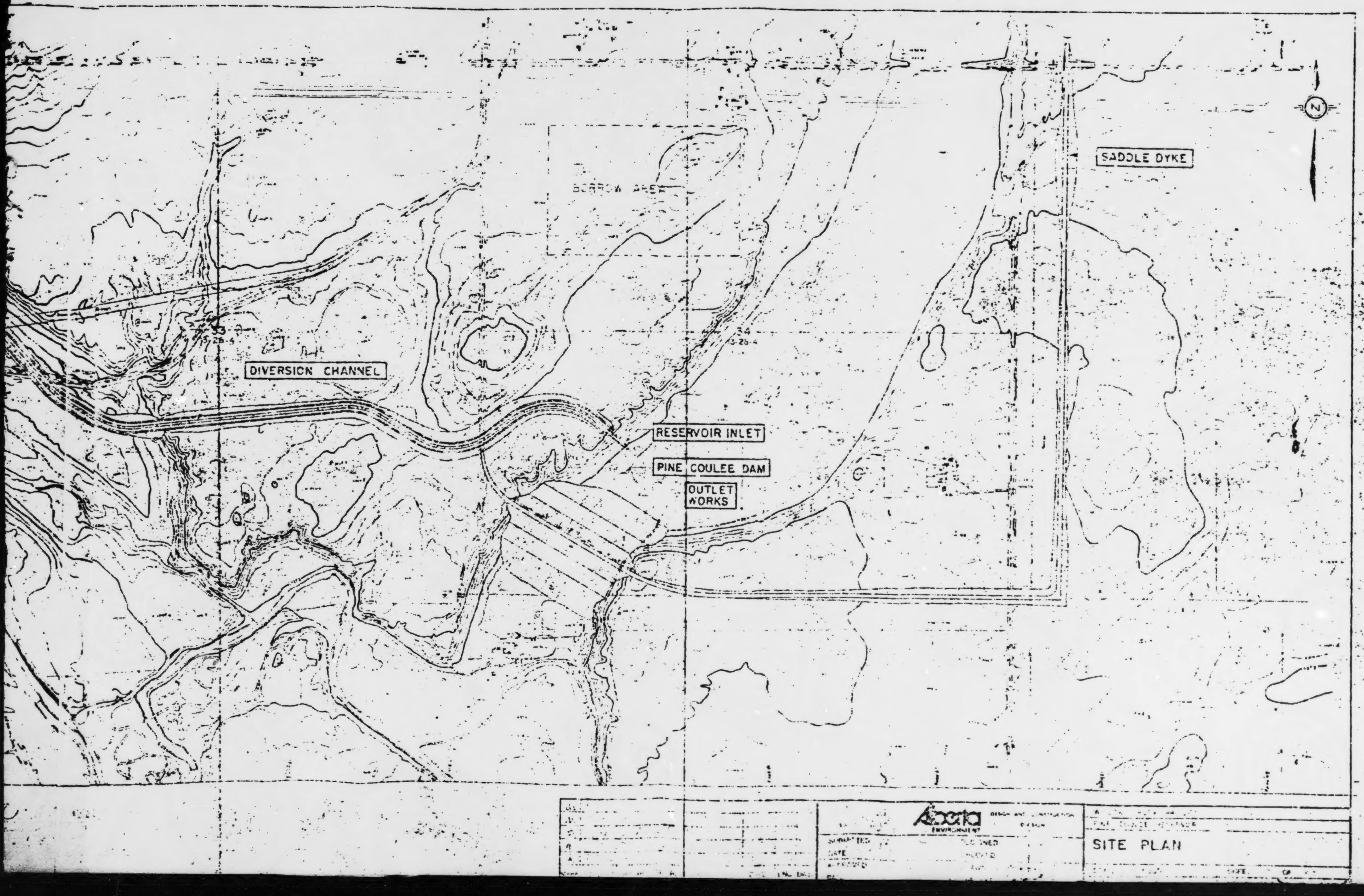
DESIGNED ....
CHECKED ....
DRAWN ..... E. Rudolf
CHECKED

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

GENERAL PLAN

SCALE 1:20000
SHEET 16 OF 43
DATE AUGUST 1985
DWG No. PICOR - F85 - 37

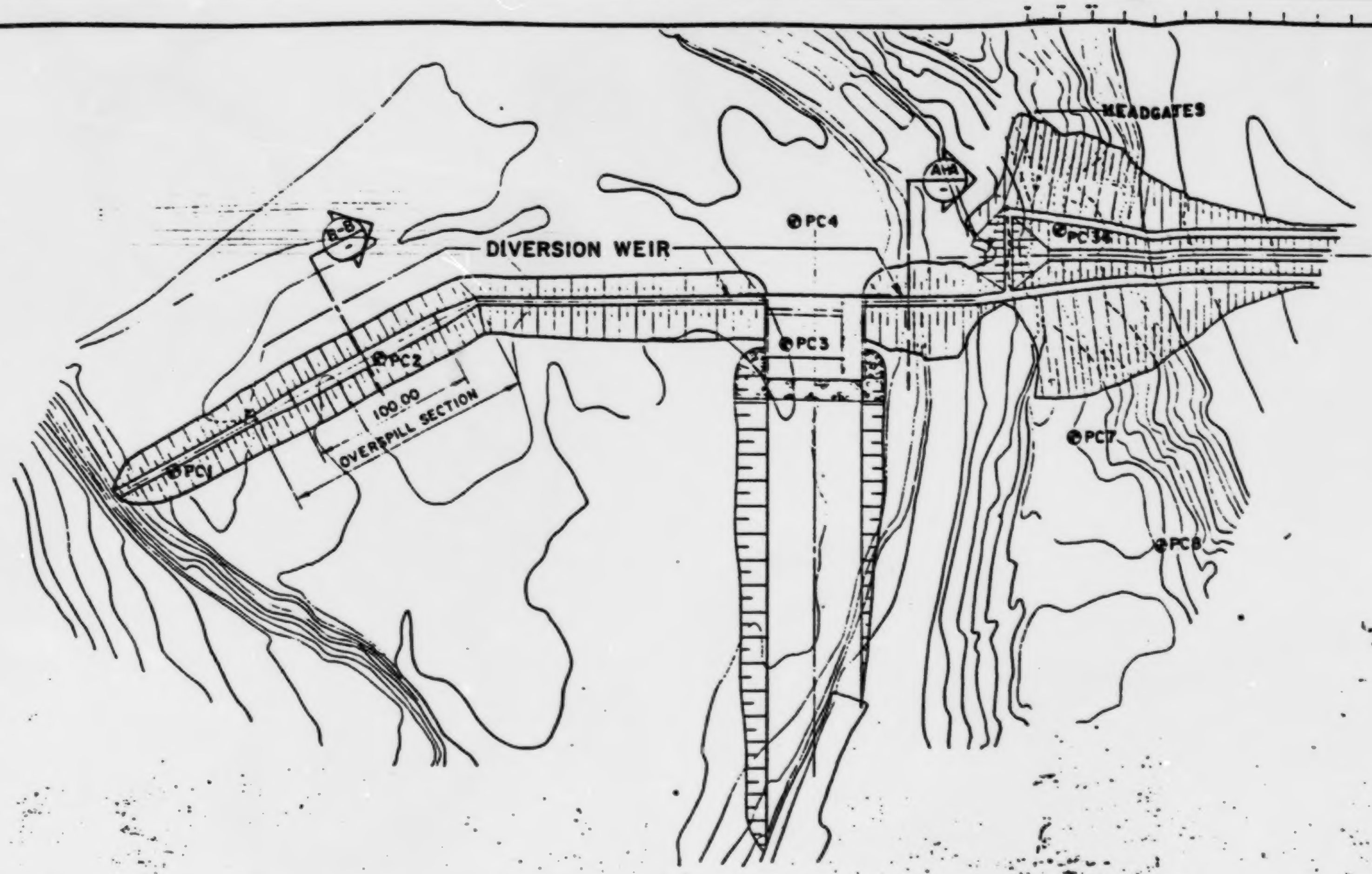

HEADGATES
DIVERSION WEIR
DIVERSION CHANNEL


SADDLE DYKE
BORROW AREA
33
34
DIVERSION CHANNEL
RESERVOIR INLET
PINE COULEE DAM
OUTLET WORKS
N
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION
SUBMITTED
DATE
APPROVED
DESIGNED
CHECKED
PINE COULEE RESERVOIR
SITE PLAN
SHEET OF

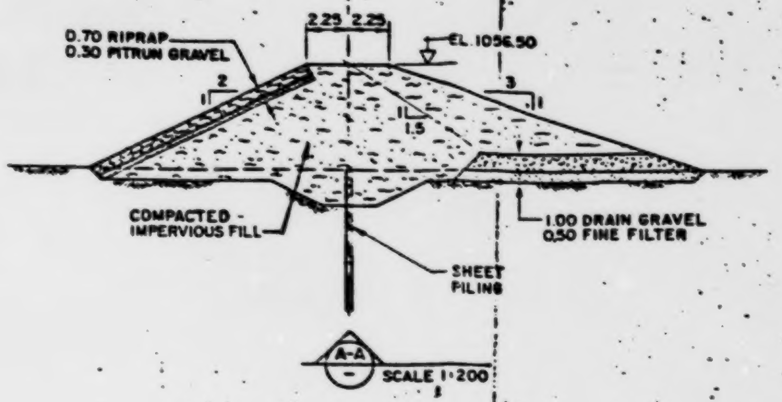
ENTRAL FILE No.

DRAWING FILE No. PICOR 0985

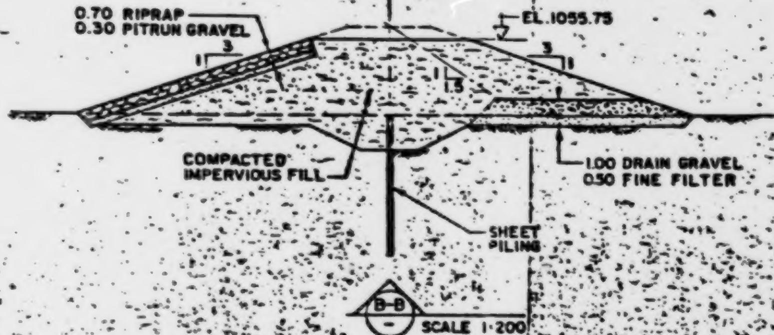
MICROFILM DATE


HEADGATES
A-A
B-B
DIVERSION WEIR
PC4
PC 5-6
PC2
PC3
PC1
100.00
OVERSPILL SECTION
PC7
PC8

PLAN
SCALE 1:2000


0.70 RIPRAP
0.30 PITRUN GRAVEL
2.25
2.25
EL. 1056.50
2
1
3
1
1.5
1
COMPACTED IMPERVIOUS FILL
1.00 DRAIN GRAVEL
0.50 FINE FILTER
SHEET PILING

A-A
SCALE 1:200


0.70 RIPRAP
0.30 PITRUN GRAVEL
EL. 1055.75
3
1
3
1
1.5
1
COMPACTED IMPERVIOUS FILL
1.00 DRAIN GRAVEL
0.50 FINE FILTER
SHEET PILING

B-B
SCALE 1:200

MARK	NATURE OF REVISION	DATE	ENG.	DWN.
△				
△				
△				
△				
△				

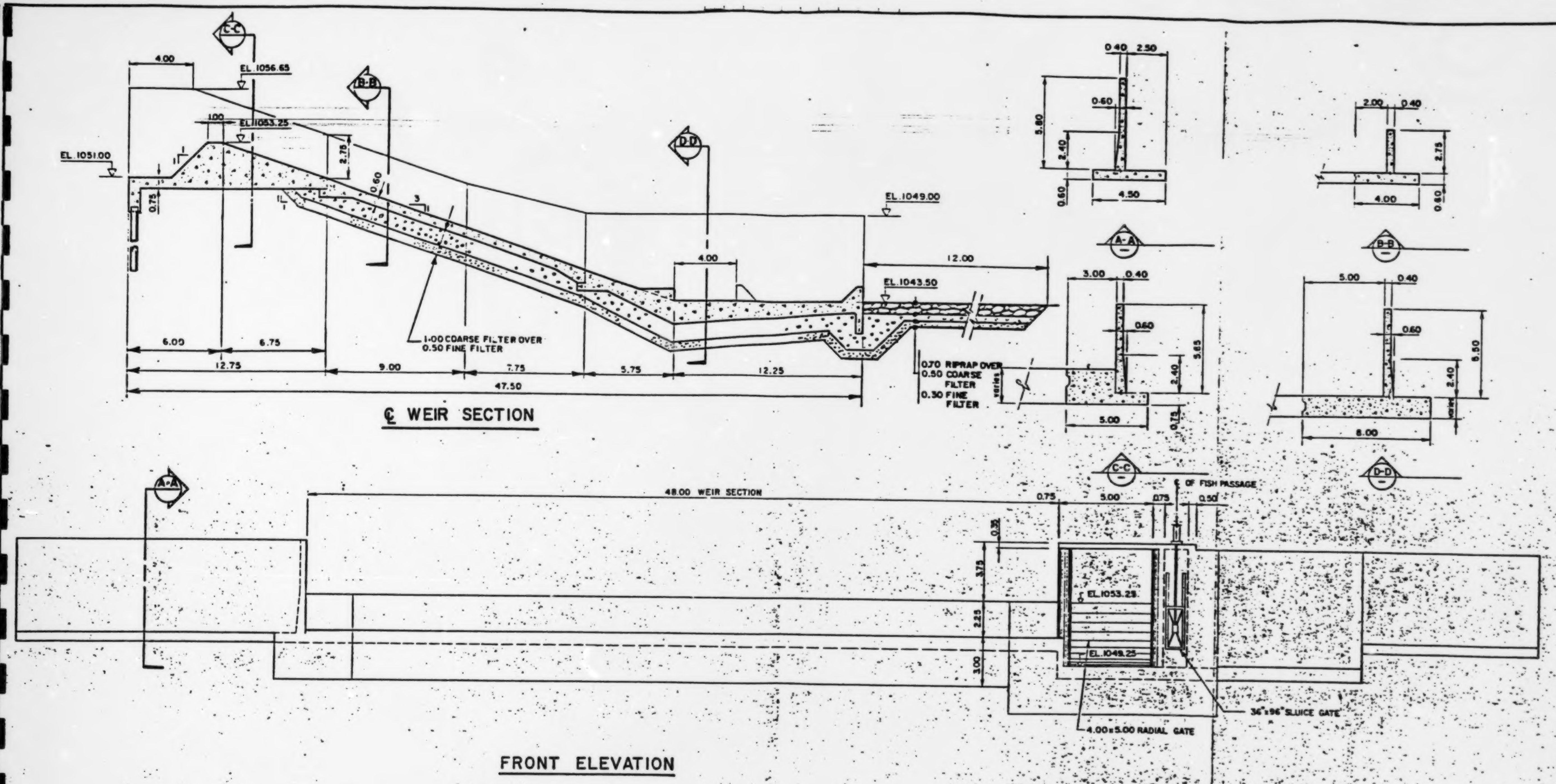
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED	DESIGNED
DATE	CHECKED
APPROVED	DRAWN C. Rudolf
DATE	CHECKED 19. ad.

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
DIVERSION WEIR

PLAN & SECTIONS

SCALE AS SHOWN
SHEET 18 OF 43
DATE SEPTEMBER, 1985
DWG No. PICOR


C-C
B-B
D-D
4.00
EL. 1056.65
1.00
EL. 1053.25
EL. 1051.00
2.75
0.60
0.75
3
1
EL. 1049.00
4.00
12.00
EL. 1043.50
1.00 COARSE FILTER OVER
0.50 FINE FILTER
6.00
6.75
12.75
9.00
7.75
5.75
12.25
47.50
0.70 RIPRAP OVER
0.50 COARSE FILTER
0.30 FINE FILTER
℄ WEIR SECTION
0.40
2.50
0.60
5.80
2.40
0.60
4.50
A-A
2.00
0.40
2.75
4.00
0.60
B-B
3.00
0.40
0.60
5.65
2.40
varies
5.00
0.75
C-C
5.00
0.40
0.60
6.50
2.40
varies
8.00
D-D
A-A
48.00 WEIR SECTION
0.75
5.00
0.75
0.50
℄ OF FISH PASSAGE
0.35
3.75
2.25
3.00
EL. 1053.25
EL. 1049.25
36"x96" SLUICE GATE
4.00x5.00 RADIAL GATE
FRONT ELEVATION

MARK	NATURE OF REVISION	DATE	ENG	DWN
△				
△				
△				
△				
△				
△				

Alberta
ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

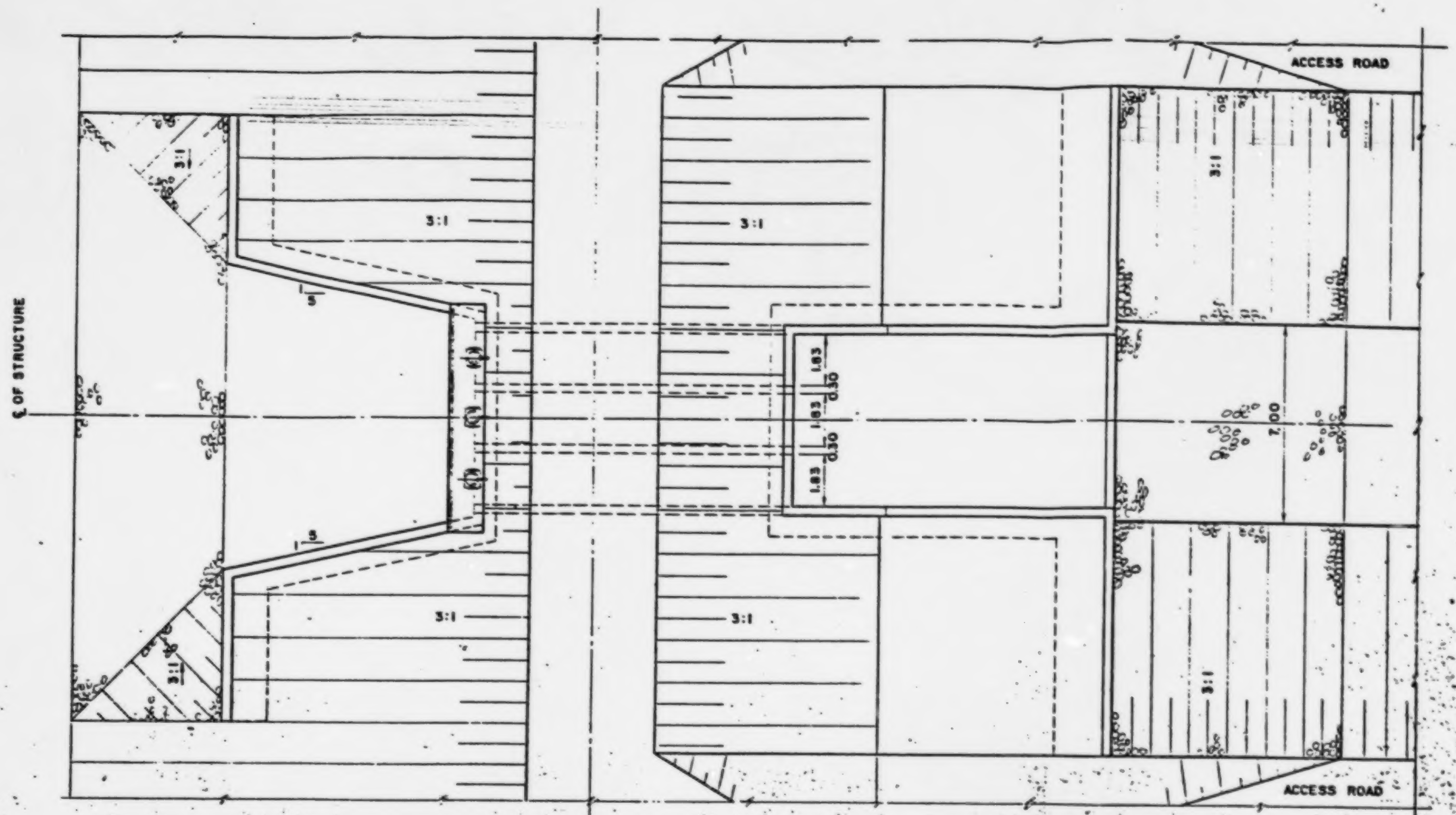
SUBMITTED ....
DATE ....
APPROVED ....

DESIGNED ....
CHECKED ....
DRAWN ....

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

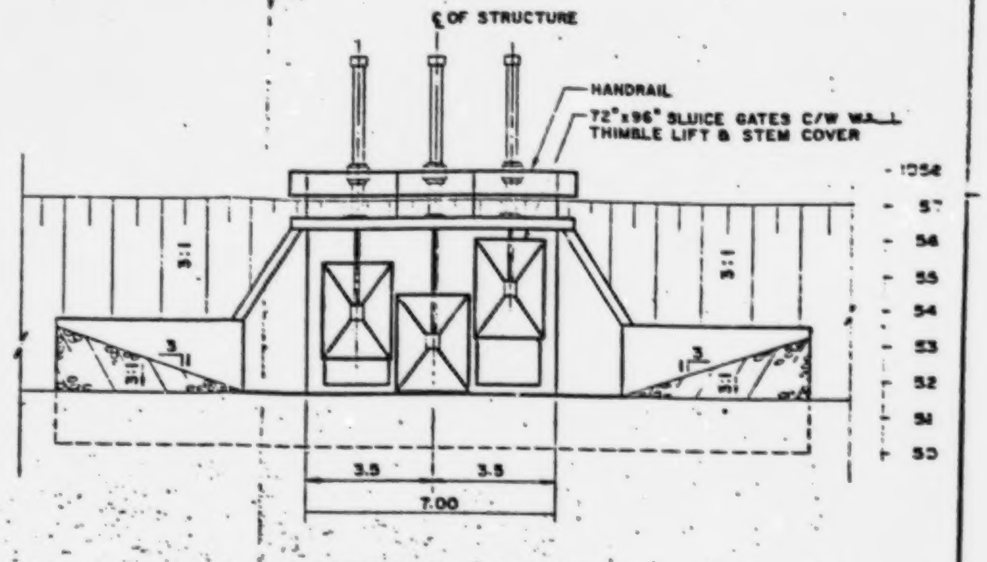
DIVERSION WEIR STRUCTURE

SCALE 1:125
SHEET 19 OF 43



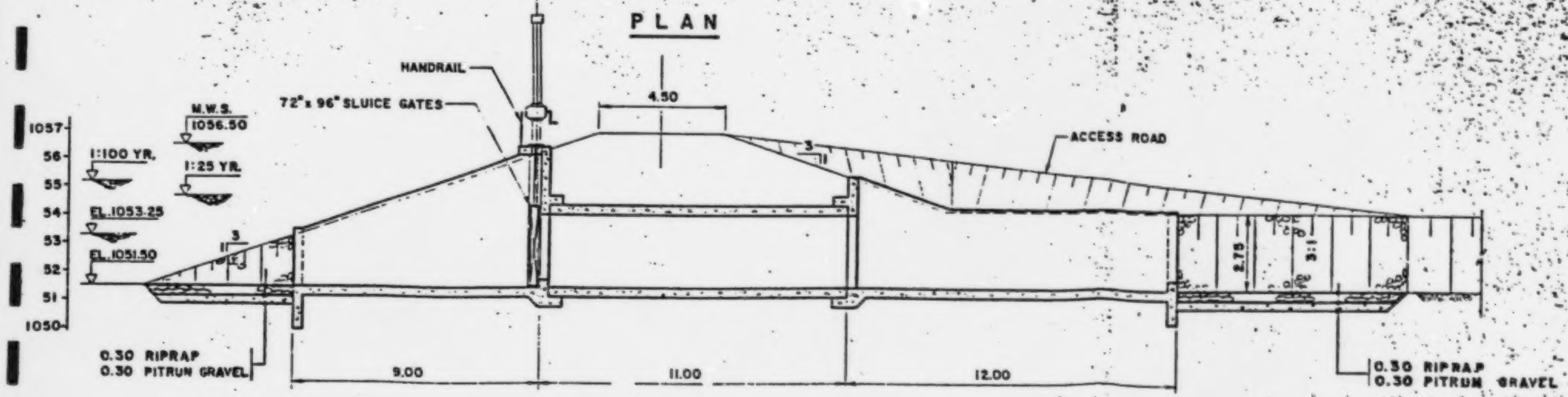
ACCESS ROAD
3:1
3:1
3:1
3:1
3:1
3:1
3:1
3:1
5
5
℄ OF STRUCTURE
1.83
0.30
1.83
0.30
1.83
7.00
ACCESS ROAD

PLAN



℄ OF STRUCTURE
HANDRAIL
72"x96" SLUICE GATES C/W WALL THIMBLE LIFT & STEM COVER
1058
57
56
55
54
53
52
51
50
3:1
3:1
3
1
3:1
3
1
3:1
3.5
3.5
7.00

UPSTREAM ELEVATION



HANDRAIL
72"x 96" SLUICE GATES
M.W.S.
1056.50
1:100 YR.
1:25 YR.
EL.1053.25
EL.1051.50
1057
56
55
54
53
52
51
1050
4.50
3
1
ACCESS ROAD
3
1
2.75
3:1
0.30 RIPRAP
0.30 PITRUN GRAVEL
9.00
11.00
12.00
0.30 RIPRAP
0.30 PITRUN GRAVEL

CENTRE LINE SECTION

MARK	NATURE OF REVISION	DATE	ENG	DWN
1				
2				
3				
4				
5				

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE ....

DESIGNED ....
CHECKED ....
DRAWN ....
CHECKED ....

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
DIVERSION CHANNEL

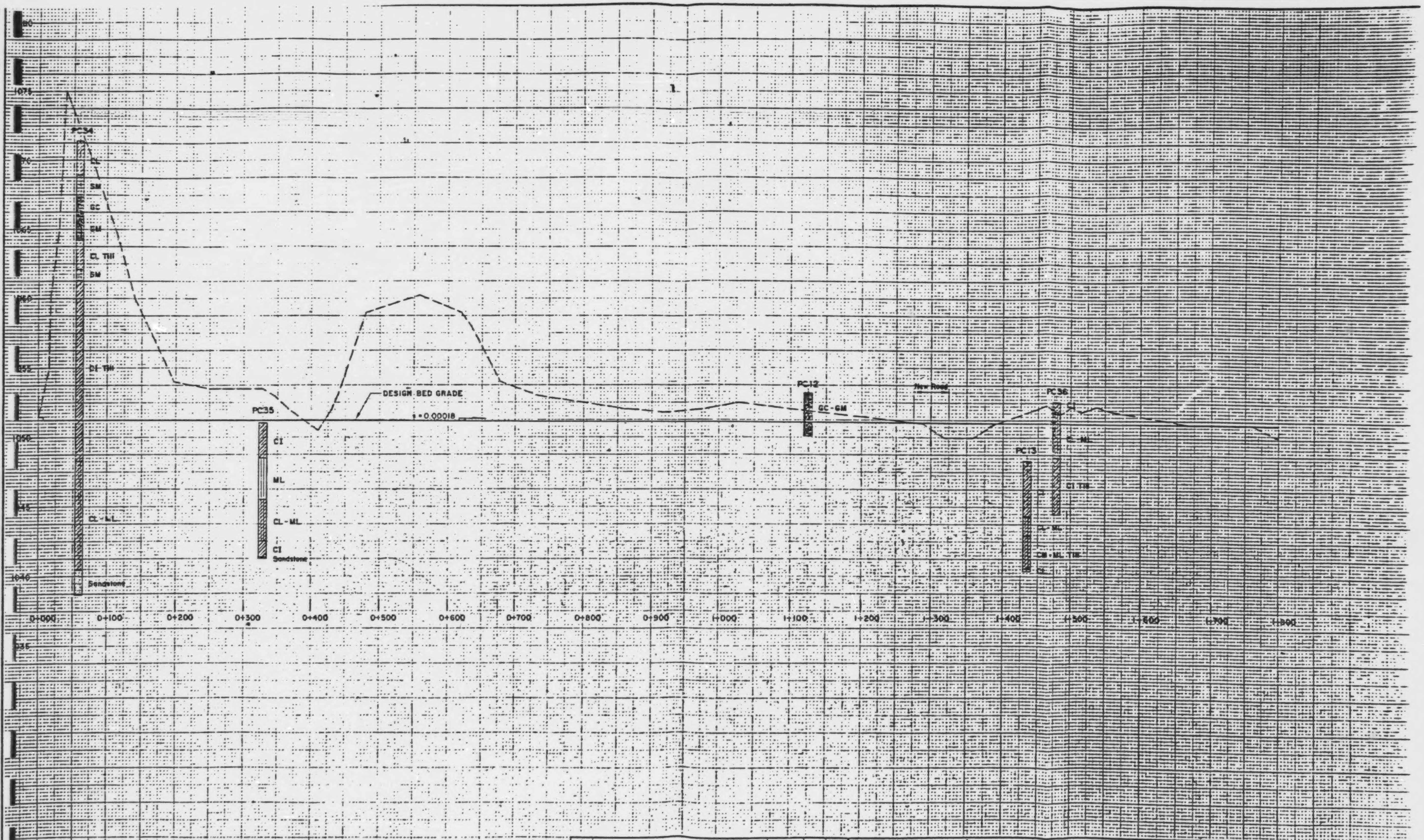
HEAD GATE STRUCTURE

SCALE 1:100
SHEET 20 OF 43
DATE AUGUST, 1985
DWG No. PICOR - E85 - 11

MICROFILM DATE

DRAWING FILE No.

CENTRAL FILE No.


1080
1075
1070
1065
1060
1055
1050
1045
1040
1035
PC34
CL
SM
GC
SM
CL TH
SM
CI TH
CL - ML
Sandstone
PC35
CI
ML
CL - ML
CI
Sandstone
DESIGN BED GRADE
s = 0.00018
PC12
GC - GM
New Road
PC36
CI
CL - ML
CI TH
PC13
CI
CL - ML
CH - ML TH
CI
0+000
0+100
0+200
0+300
0+400
0+500
0+600
0+700
0+800
0+900
1+000
1+100
1+200
1+300
1+400
1+500
1+600
1+700
1+800

MARK	NATURE OF REVISION	DATE	ENG	DWN
E				
D				
C				
B				
A				

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....

DATE ....

APPROVED ....

DATE ....

DESIGNED ....

CHECKED ....

DRAWN .... C.W. Rudolf

CHECKED .... 19. W.

WILLOW CREEK PROJECT

PINE COULEE RESERVOIR

DIVERSION CHANNEL

STRATIGRAPHIC PROFILE

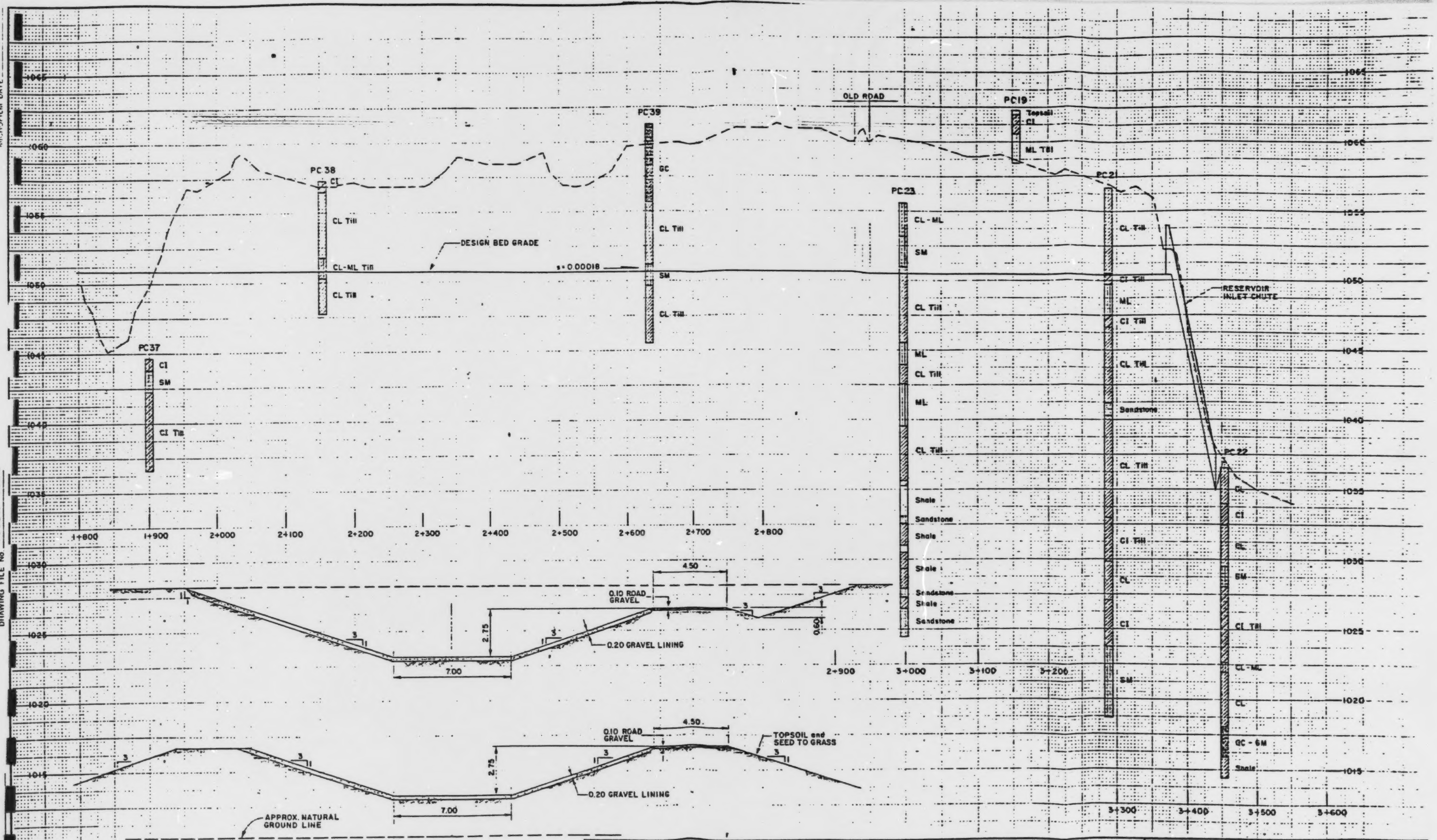
STA. 0+000 TO STA. 1+800

SCALE Horiz. 1:2500; Vert. 1:125 SHEET 21 OF 43

DATE SEPTEMBER, 1985 DWG No PICOR - E85 - 12

MICROFILM DATE

DRAWING FILE No. PICOR 1385


1065
1060
1055
1050
1045
1040
1035
1030
1025
1020
1015
1010
OLD ROAD
PC19
Topsoil
CL
ML Till
PC39
GC
CL Till
SM
CL Till
PC 38
CL
CL Till
CL-ML Till
CL Till
DESIGN BED GRADE
s = 0.00018
PC23
CL - ML
SM
CL Till
ML
CL Till
ML
CL Till
Shale
Sandstone
Shale
Shale
Sandstone
Shale
Sandstone
PC21
CL Till
CL Till
ML
CL Till
CL Till
Sandstone
CL Till
CL Till
CL
CI
SM
RESERVOIR INLET CHUTE
PC22
CL
CI
CL
SM
CI Till
CL - ML
CL
GC - SM
Shale
PC37
CI
SM
CI Till
1+800
1+900
2+000
2+100
2+200
2+300
2+400
2+500
2+600
2+700
2+800
2+900
3+000
3+100
3+200
3+300
3+400
3+500
3+600
4.50
0.10 ROAD GRAVEL
3
1
2.75
0.60
0.20 GRAVEL LINING
7.00
4.50
0.10 ROAD GRAVEL
TOPSOIL and SEED TO GRASS
3
1
2.75
0.20 GRAVEL LINING
7.00
APPROX. NATURAL GROUND LINE

TYPICAL
CROSS-SECTIONS

MARK	NATURE OF REVISION	DATE	ENG. DWN.
E			
D			
C			
B			
A			

Alberta
ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE ....

DESIGNED ....
CHECKED ....
DRAWN .... L. Kudolt
CHECKED .... 18.

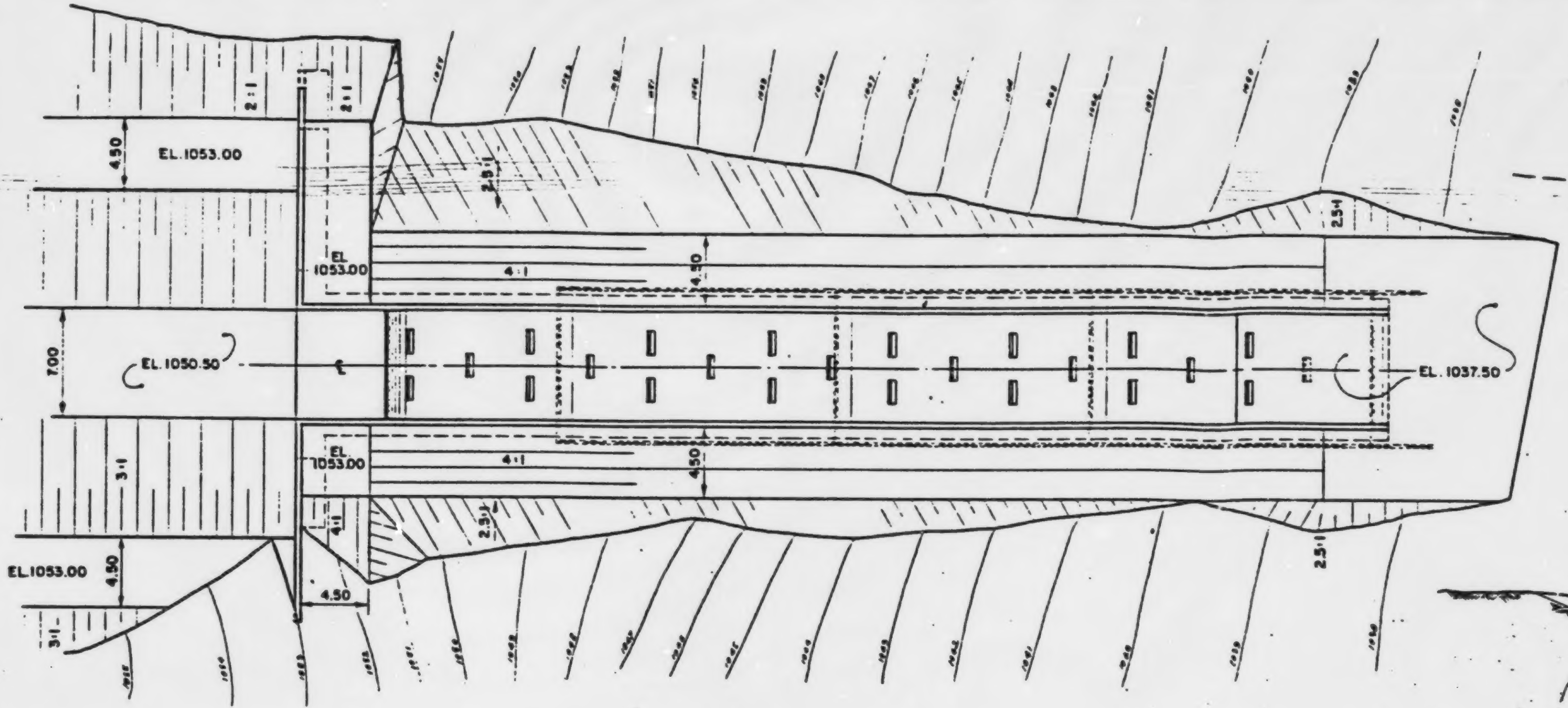
WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
DIVERSION CHANNEL
STRATIGRAPHIC PROFILE
STA. 1+800 TO STA. 3+550

SCALE Horiz 1:2500; Vert 1:125
SHEET 22 OF 43
DATE SEPTEMBER, 1985
DWG No. PICOR - E85 - 13

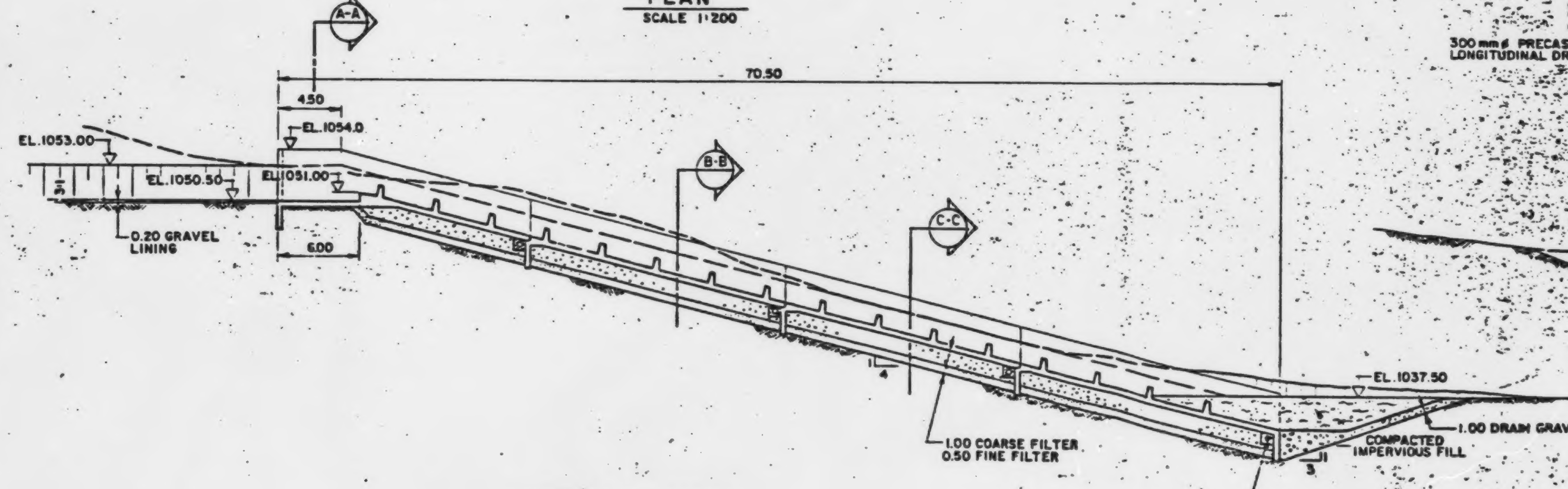
MICROFILM DATE

DRAWING FILE No. PICOR-485

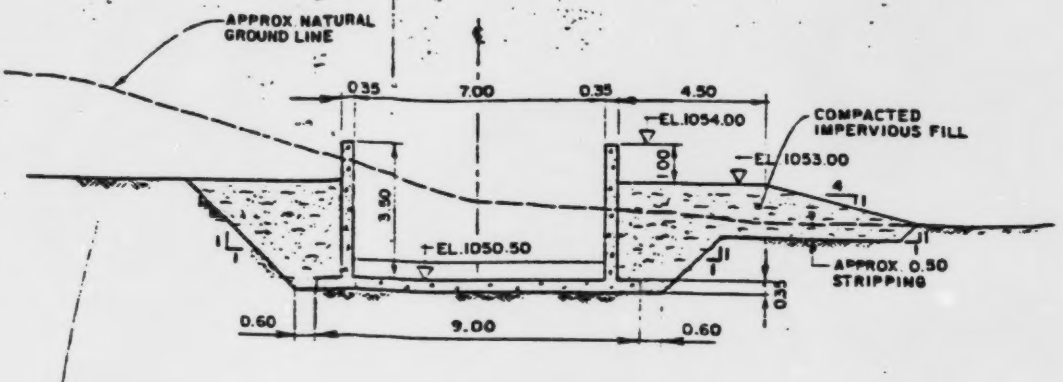
CENTRAL FILE No.


EL.1053.00
4.50
2:1
2.5:1
EL 1053.00
4:1
4.50
7.00
EL.1050.50
℄
EL.1037.50
3:1
4:1
EL.1053.00
4.50
2.5:1

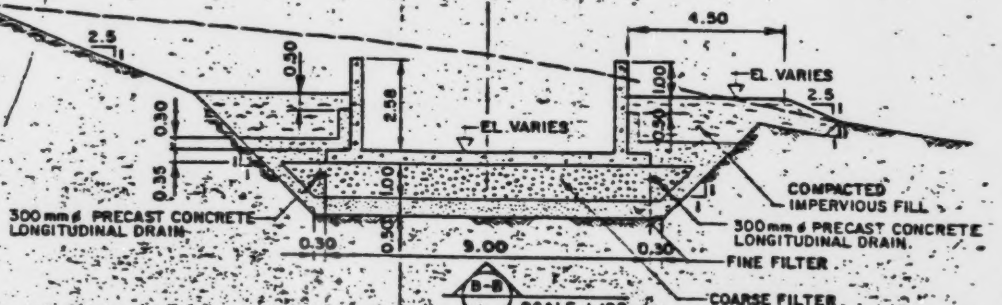
PLAN
SCALE 1:200


A-A
70.50
4.50
EL.1054.0
EL.1053.00
EL.1050.50
EL.1051.00
3:1
0.20 GRAVEL LINING
6.00
B-B
C-C
4
1.00 COARSE FILTER
0.50 FINE FILTER
EL.1037.50
1.00 DRAIN GRAVEL
COMPACTED IMPERVIOUS FILL
3
200 mm ø PERFORATED CROSS DRAIN PIPE

CENTERLINE PROFILE
SCALE 1:200

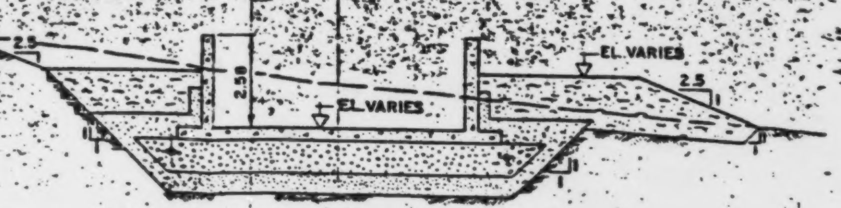

APPROX NATURAL GROUND LINE
℄
0.35
7.00
0.35
4.50
EL.1054.00
COMPACTED IMPERVIOUS FILL
EL.1053.00
1.00
3.50
4
EL.1050.50
APPROX 0.50 STRIPPING
0.35
0.60
9.00
0.60

A-A SCALE 1:100


2.5
0.50
2.58
4.50
1.00
EL. VARIES
2.5
0.30
EL. VARIES
0.30
0.35
1.00
COMPACTED IMPERVIOUS FILL
300 mm ø PRECAST CONCRETE LONGITUDINAL DRAIN
300 mm ø PRECAST CONCRETE LONGITUDINAL DRAIN.
0.30
0.50
9.00
0.30
FINE FILTER
COARSE FILTER

B-B SCALE 1:100

NOTE:
ALL DIMENSION ARE TYPICAL FOR SECTIONS B-B and C-C EXCEPT AS SHOWN.


2.5
2.58
EL. VARIES
EL. VARIES
2.5

C-C SCALE 1:100

MARK	NATURE OF REVISION	DATE	ENG	DWN
△				
△				
△				
△				
△				

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE ....

DESIGNED ....
CHECKED ....
DRAWN ....
CHECKED ....

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
RESERVOIR INLET

PLAN and PROFILE

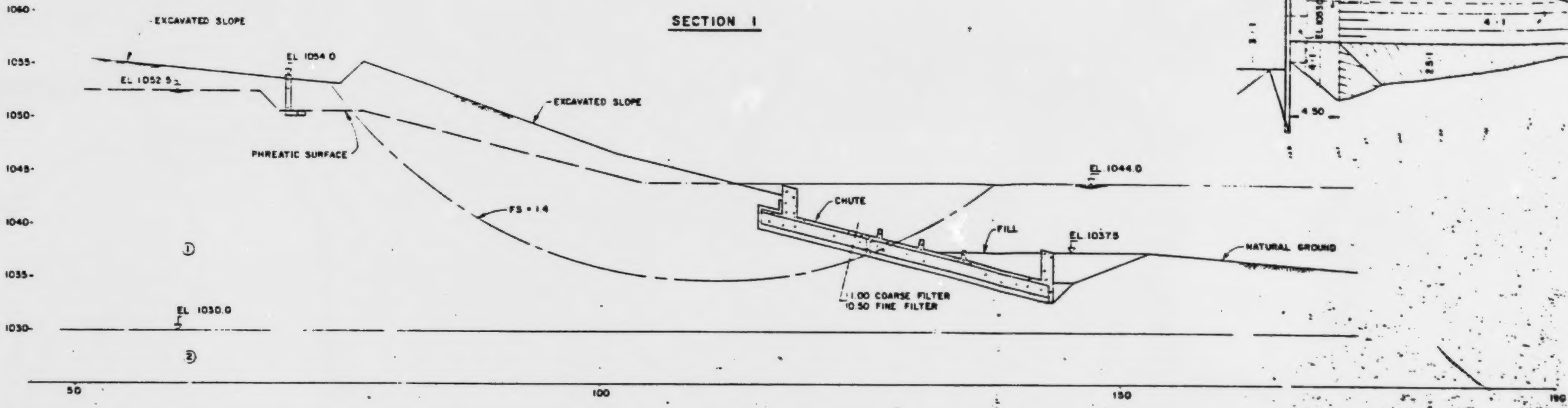
SCALE AS SHOWN
SHEET 23 OF 43
DATE SEPTEMBER
DWG No PICOR - E83 - 14

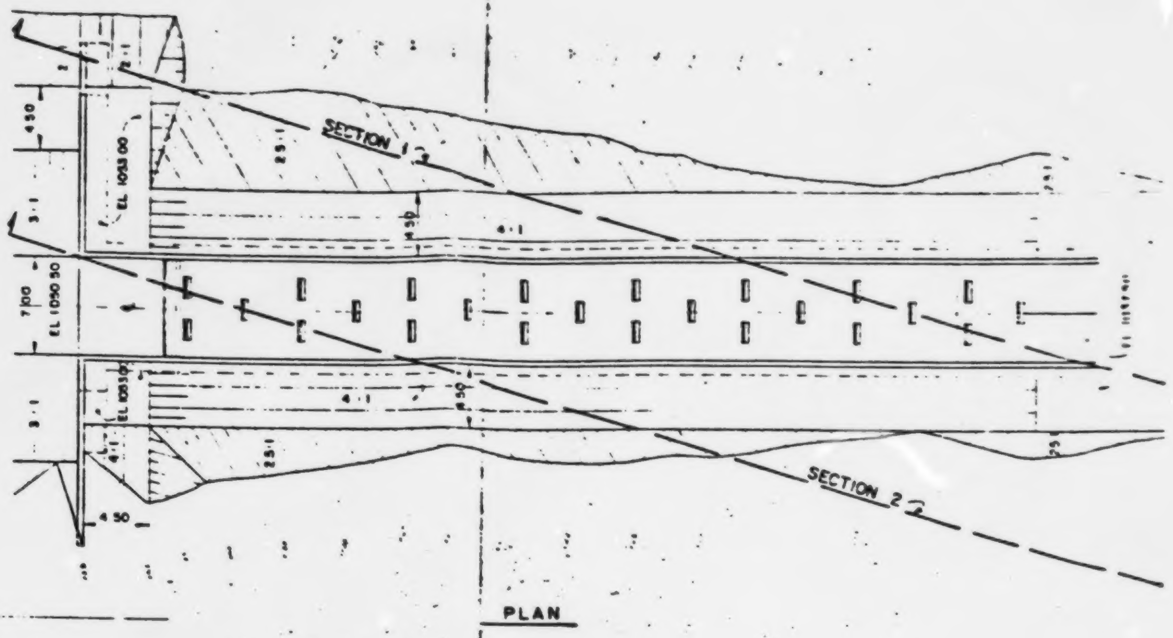
MICROFILM DATE

DRAWING FILE No. PICOR 4085

CENTRAL FILE No.

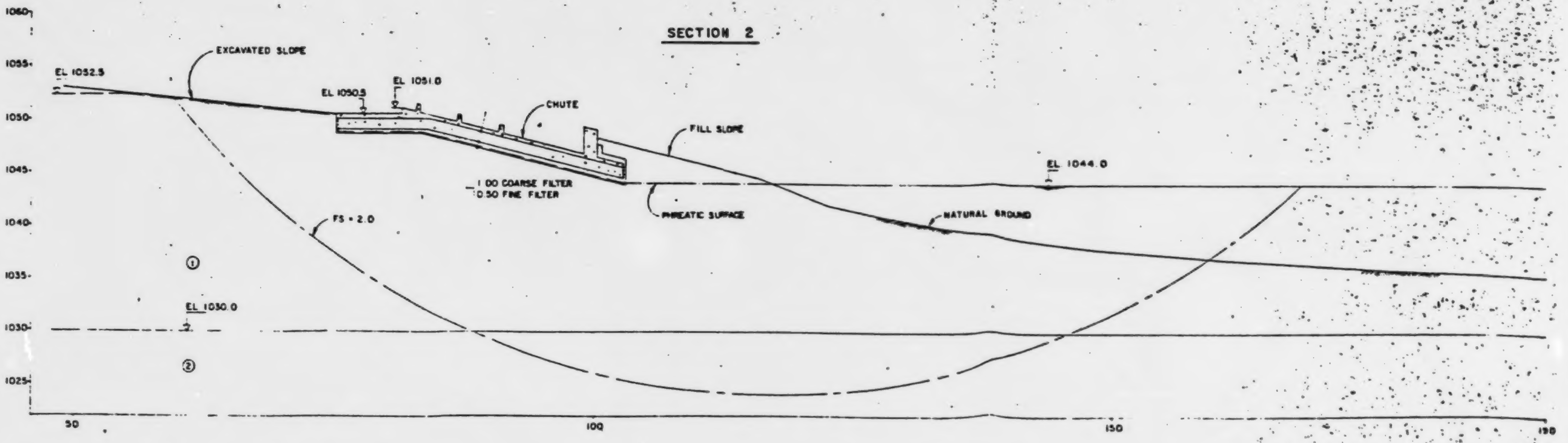
SECTION 1


1060
1055
1050
1045
1040
1035
1030
EXCAVATED SLOPE
EL 1054.0
EL 1052.5
EXCAVATED SLOPE
PHREATIC SURFACE
FS = 1.4
CHUTE
FILL
EL 1044.0
EL 1037.5
NATURAL GROUND
1.00 COARSE FILTER
0.50 FINE FILTER
EL 1030.0
①
②
50
100
150
190


SECTION 1
SECTION 2
EL 1053.00
EL 1050.50
EL 1053.00
4.50
7.00
4.50
3:1
4:1
2.5:1

PLAN

SECTION 2


1060
1055
1050
1045
1040
1035
1030
1025
EL 1052.5
EXCAVATED SLOPE
EL 1050.5
EL 1051.0
CHUTE
FILL SLOPE
EL 1044.0
1.00 COARSE FILTER
0.50 FINE FILTER
PHREATIC SURFACE
NATURAL GROUND
FS = 2.0
EL 1030.0
①
②
50
100
150
190

NOTE

The combined average minimum factor of safety calculated for the overall chute structure is 1.5

ZONE	SOIL PROPERTIES γ (t/m³)	ϕ'	C' (Kpa)
① CL (TILL)	2.1	25	10
② CI	2.1	18	9.0

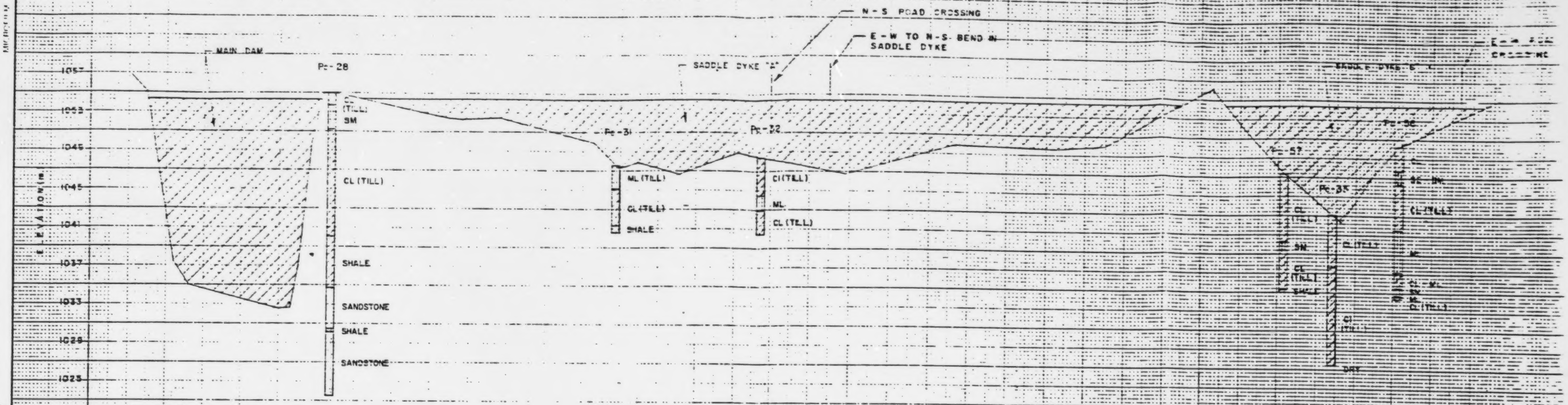
MARK	NATURE OF REVISION	DATE	ENG	DWN
1				
2				
3				
4				
5				

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE ....
DESIGNED ....
CHECKED ....
DRAWN F. F.
CHECKED John Wong

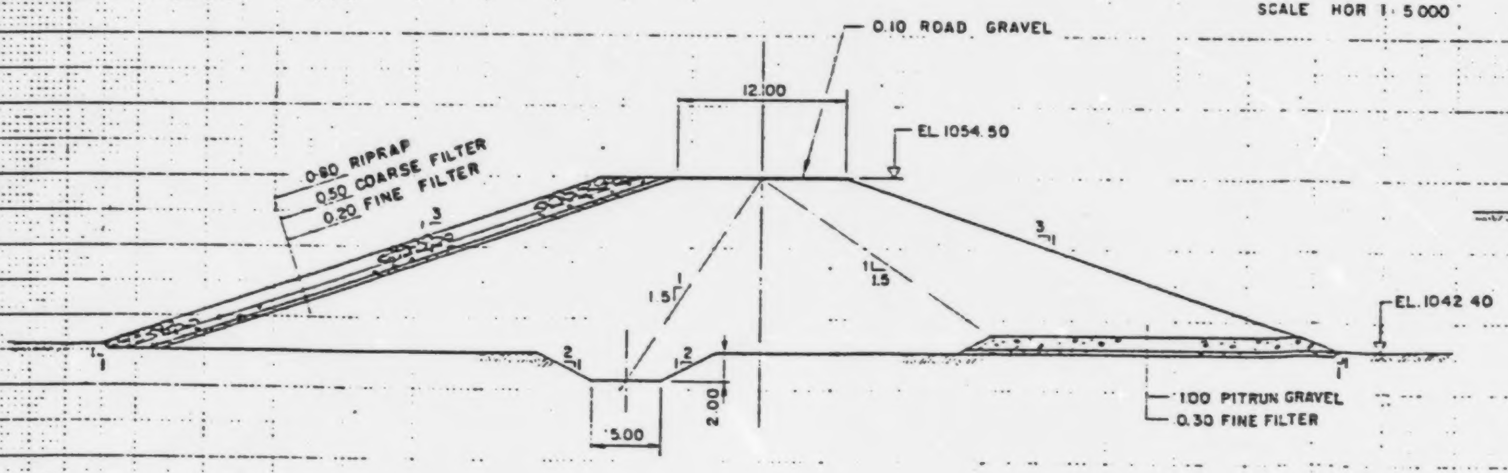
WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
RESERVOIR INLET CHUTE
STABILITY ANALYSIS
DRAWDOWN CONDITION
SCALE 1 : 250
SHEET 24 OF 43
DATE OCT. 1985
DWG No. PICOR - 585 - 40

MICROFILM DATE
DRAWING FILE No. PICOR 1885
CENTRAL FILE No.

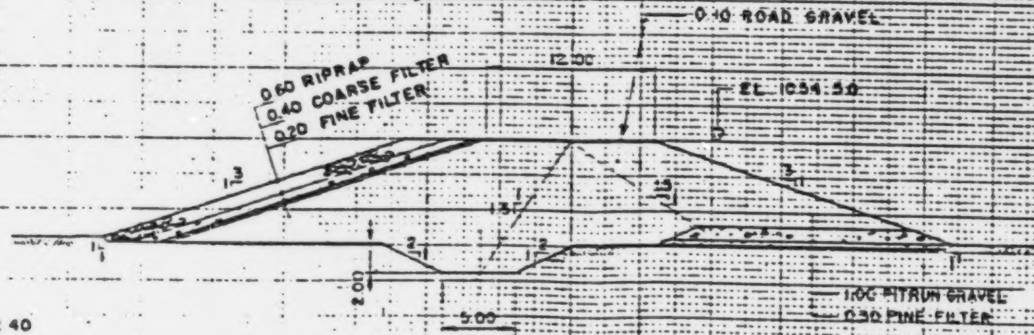

N-S ROAD CROSSING
E-W TO N-S BEND IN SADDLE DYKE
E-W ROAD CROSSING
MAIN DAM
Pz-28
SADDLE DYKE "A"
SADDLE DYKE "B"
ELEVATION (m)
1057
1053
1049
1045
1041
1037
1033
1029
1025
CL (TILL)
SM
CL (TILL)
SHALE
SANDSTONE
SHALE
SANDSTONE
Pz-31
ML (TILL)
CL (TILL)
SHALE
Pz-32
CL (TILL)
ML
CL (TILL)
Pz-37
Pz-36
Pz-35
CL (TILL)
SM
CL (TILL)
SHALE
CL (TILL)
DRY
CL
SC SM
CL (TILL)
ML
CL ML
SC
ML
CL (TILL)

SEE SHEET 17 FOR LOCATION OF SADDLE DYKE

PROFILE
SCALE HOR 1:5000


0.10 ROAD GRAVEL
12.00
EL. 1054.50
0.80 RIPRAP
0.50 COARSE FILTER
0.20 FINE FILTER
3
1
1.5
1
1
1.5
3
1
EL. 1042.40
1
1
2
1
1
2
2.00
5.00
1.00 PITRUN GRAVEL
0.30 FINE FILTER

12 m. HIGH SECTION
SCALE 1:250


0.10 ROAD GRAVEL
12.00
EL. 1054.50
0.60 RIPRAP
0.40 COARSE FILTER
0.20 FINE FILTER
3
1
1.5
1
1.5
1
3
1
1
1
2
1
1
2
2.00
5.00
1.00 PITRUN GRAVEL
0.30 FINE FILTER

7m HIGH SECTION
SCALE 1:250

MARK	NATURE OF REVISION	DATE	ENG	DWN
1				
2				
3				
4				
5				

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

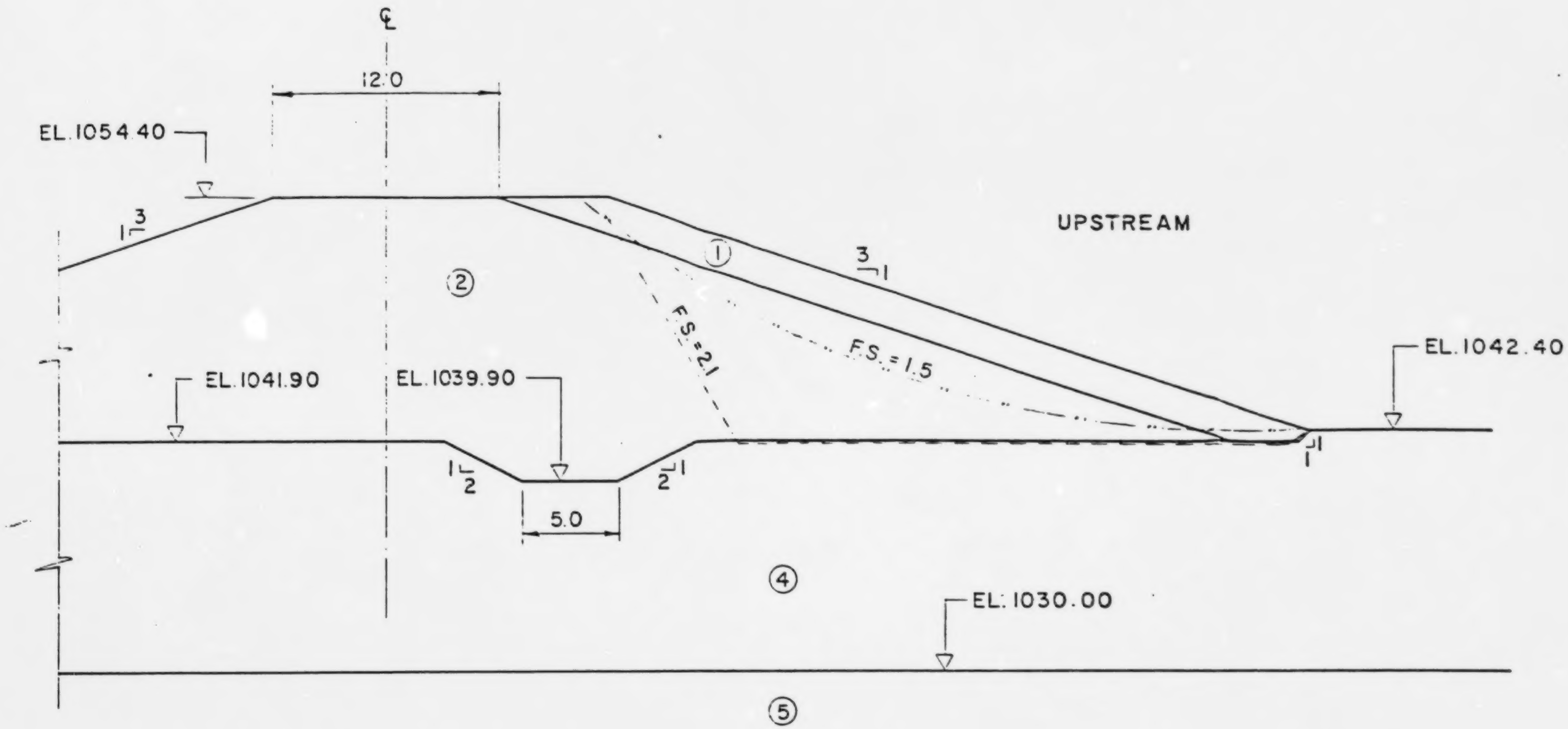
SUBMITTED ....
DATE ....
APPROVED ....
DATE ....
DESIGNED ....
CHECKED ....
DRAWN
CHECKED p.s.

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

SADDLE DYKE

SCALE HOR. 1:5000
DATE SEPT. 1985
SHEET 25 OF 43
DWG No. PICOR - E55 - 5

P1-560×860

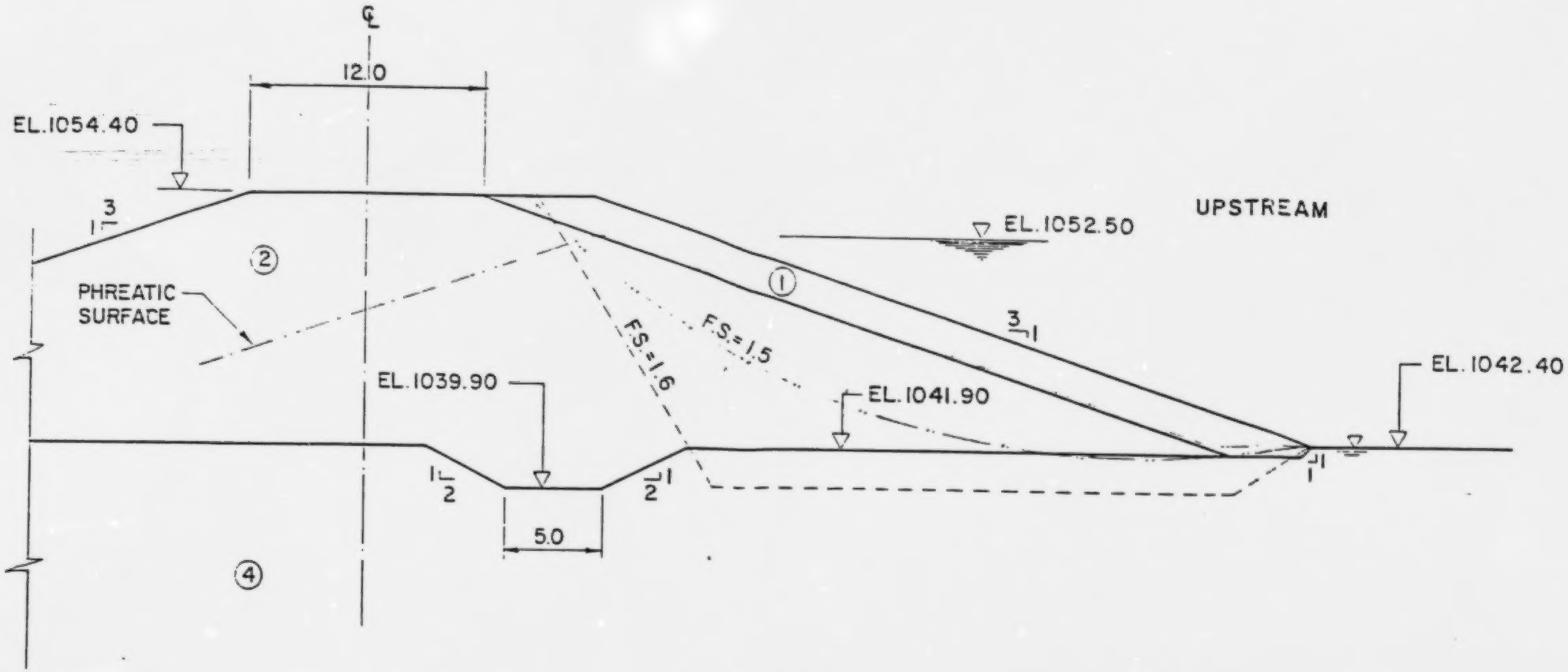

℄
12.0
EL. 1054.40
1 3
3 1
UPSTREAM
①
②
F.S. = 2.1
F.S. = 1.5
EL. 1042.40
EL. 1041.90
EL. 1039.90
1 2
2 1
1 1
5.0
④
EL. 1030.00
⑤

ZONE		SOIL PROPERTIES			
		γ_{bulk} (t/m³)	Ø'	c'(KPa)	r_u
①	SLOPE PROTECTION	2.1	32	0	0
②	IMPERVIOUS FILL	2.0	28	3.45	0.25
④	FOUNDATION ABOVE WATER TABLE	2.1	25	10	0.20
⑤	FOUNDATION BELOW WATER TABLE	2.1	25	10	0.50

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

SUBMITTED	DESIGNED	WILLOW CREEK PROJECT
DATE	CHECKED	PINE COULEE RESERVOIR
		SADDLE DYKE STABILITY ANALYS END OF CONSTRUCTION U/S SLOPE
APPROVED	DRAWN B. K.	SCALE SHEET 26 OF 43
DATE	CHECKED	DATE DRAWING No.

DRAWING FILE No. PICOR 4385


℄
12.0
EL.1054.40
3
1
UPSTREAM
EL.1052.50
②
①
PHREATIC SURFACE
3
1
F.S.=1.5
F.S.=1.6
EL.1042.40
EL.1039.90
EL.1041.90
1
2
2
1
1
1
5.0
④

ZONE / SOIL PROPERTIES

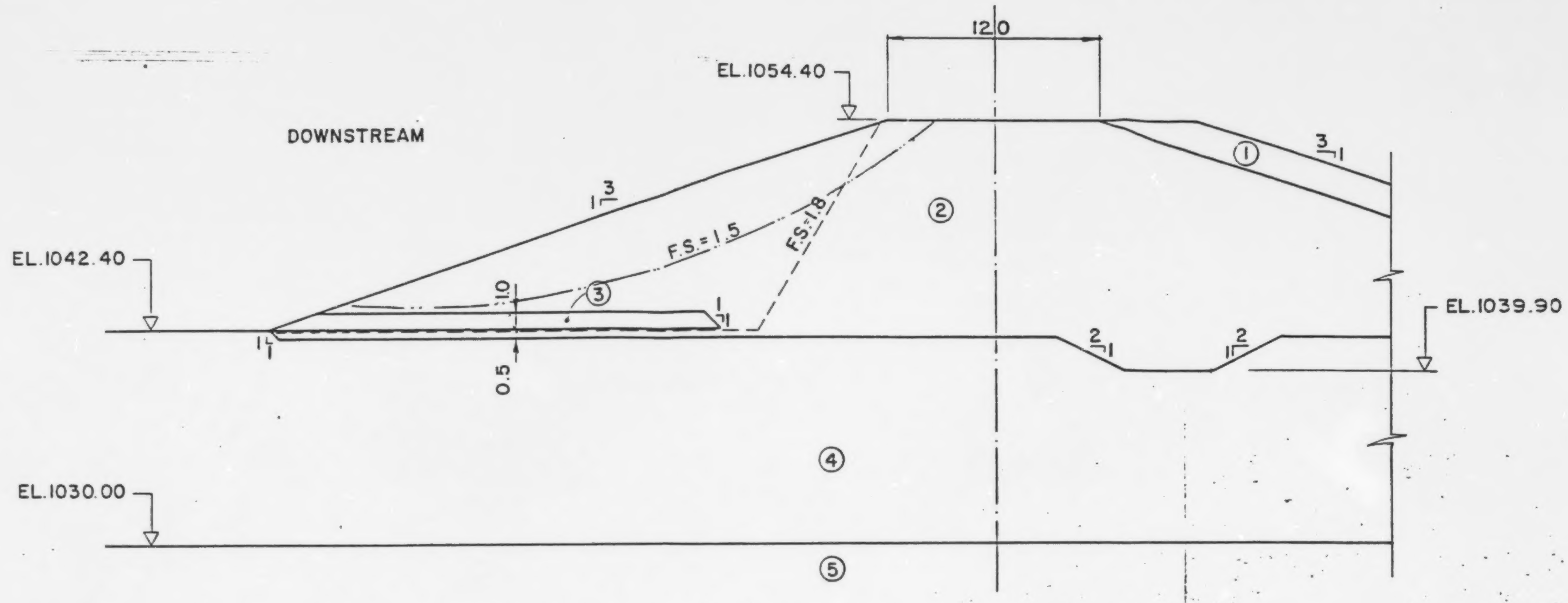
ZONE		$\gamma_{bulk}(t/m^3)$	Ø'	c'(KPa)
①	SLOPE PROTECTION	2.1	32	0
②	IMPERVIOUS FILL	2.0	28	3.45
④	FOUNDATION	2.1	25	10

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED		DESIGNED	
DATE		CHECKED	
APPROVED		DRAWN	B.K.
DATE		CHECKED	/

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
SADDLE DYKE STABILITY ANALYSIS
DRAWDOWN CONDITION
U/S SLOPE
SCALE
SHEET 27 OF 43
DATE
DRAWING No

P3-280×430



12.0
EL.1054.40
DOWNSTREAM
1
3
3
1
①
②
F.S.=1.5
F.S.=1.8
EL.1042.40
1.0
③
1
1
1
1
0.5
EL.1039.90
2
1
1
2
④
EL.1030.00
⑤

ZONE		SOIL PROPERTIES			
		γ_{bulk} (t/m³)	Ø'	c'(KPa)	r_u
①	SLOPE PROTECTION	2.1	32	0	0
②	IMPERVIOUS FILL	2.0	28	3.45	0.25
③	DRAIN GRAVEL	1.8	32	0	0
④	FOUNDATION ABOVE WATER TABLE	2.1	25	10	0.20
⑤	FOUNDATION BELOW WATER TABLE	2.1	25	10	0.50

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

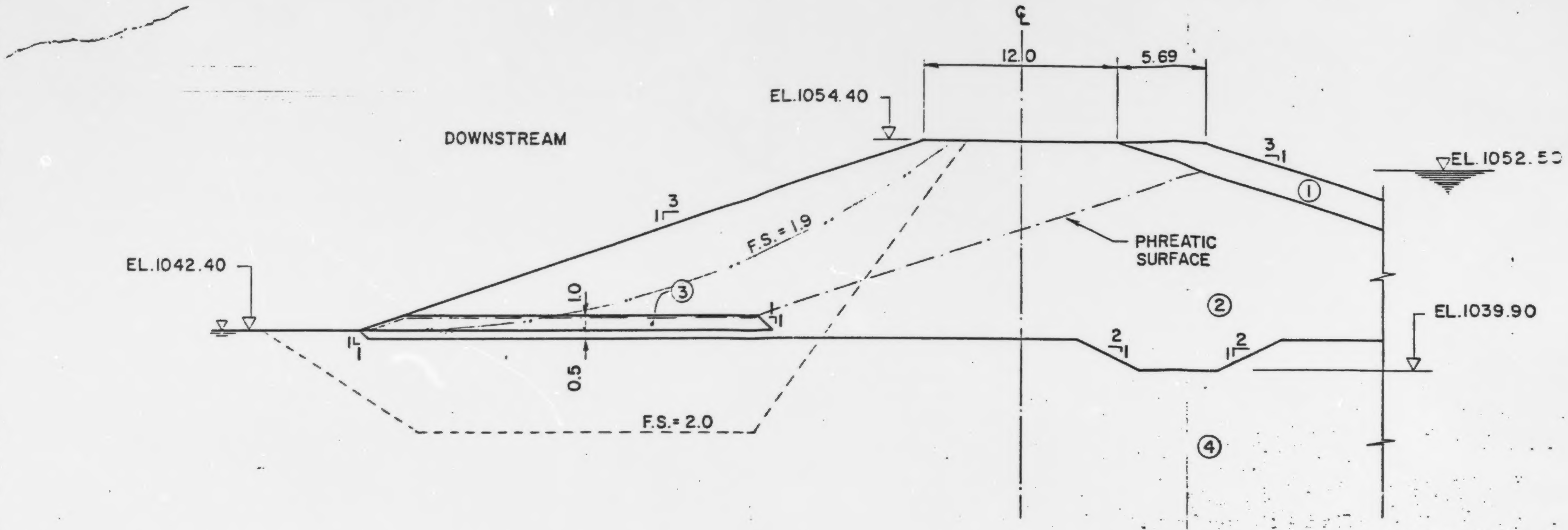
SUBMITTED
DATE
APPROVED
DATE
DESIGNED
CHECKED
DRAWN B. K.
CHECKED

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

SADDLE DYKE STABILITY ANALYSIS END OF CONSTRUCTION D/S SLOPE

SCALE
SHEET 28 OF 43
DATE
DRAWING

DRAWING FILE No PICOR 4585


℄
12.0
5.69
EL.1054.40
DOWNSTREAM
3
1
EL.1052.5
①
3
1
F.S. = 1.9
PHREATIC SURFACE
EL.1042.40
1.0
③
1
1
②
EL.1039.90
1
1
2
1
2
1
0.5
F.S. = 2.0
④

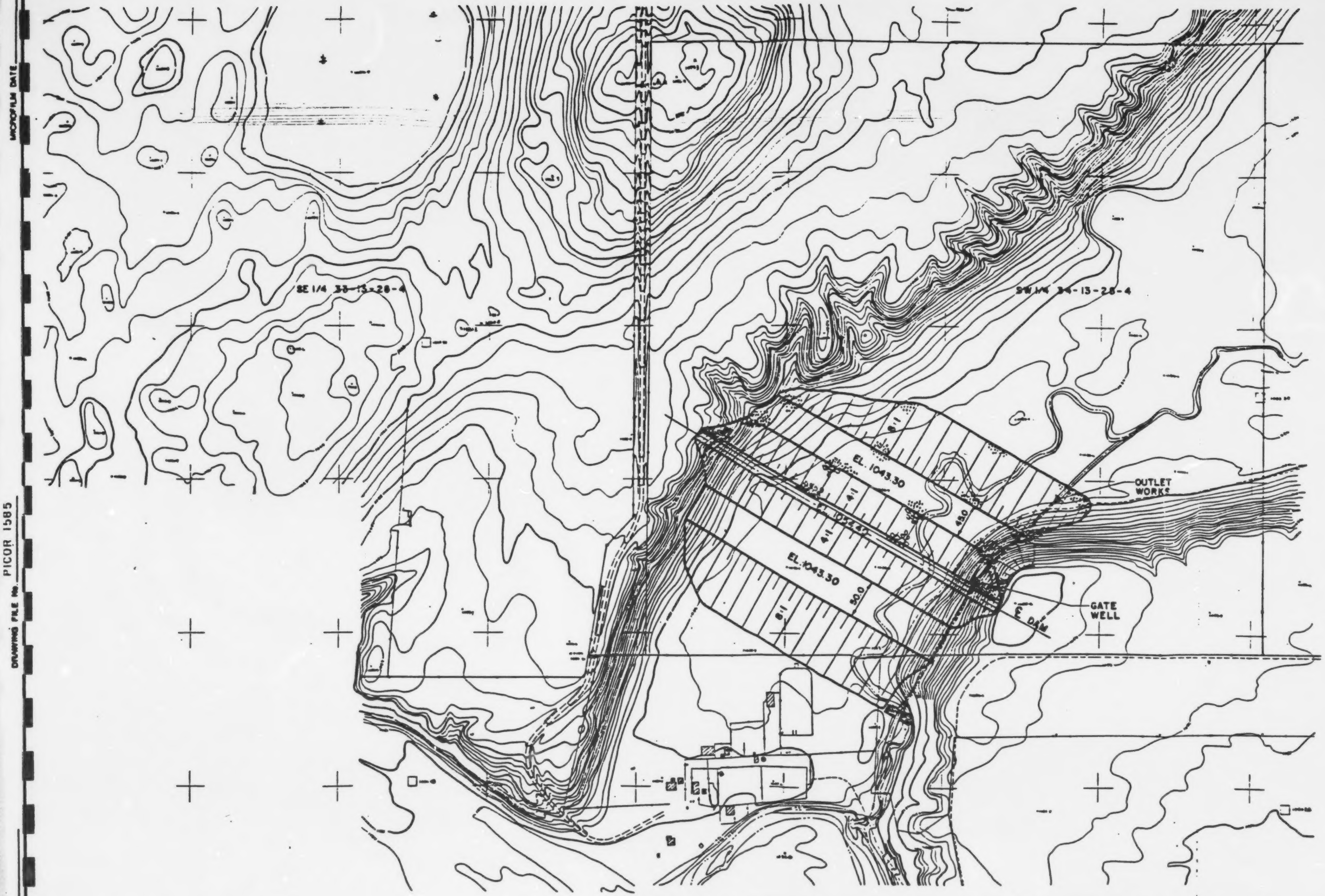
ZONE		SOIL PROPERTIES		
		$\gamma_{bulk}(t/m^3)$	Ø'	c'(KPa)
①	SLOPE PROTECTION	2.1	32	0
②	IMPERVIOUS FILL	2.0	28	3.45
③	DRAIN GRAVEL	2.1	32	0
④	FOUNDATION	2.1	25	10

Alberta ENVIRONMENT	DESIGN AND CONSTRUCTION DIVISION	WILLOW CREEK PROJECT
		PINE COULEE RESERVOIR
SUBMITTED	DESIGNED	SADDLE DYKE STABILITY ANALYSIS STEADY STATE SEEPAGE D/S SLOPE
DATE	CHECKED	
APPROVED	DRAWN B. K.	SCALE · SHEET 29 OF 43
DATE	CHECKED	DATE · DRAWING No

MICROFILM DATE

DRAWING FILE No. PICOR 1585

CENTRAL FILE No.


SE 1/4 33-13-28-4
SW 1/4 34-13-28-4
EL. 1043.30
EL. 1052.40
EL. 1043.30
4:1
4:1
6:1
6:1
45.0
30.0
OUTLET WORKS
GATE WELL
℄ DAM

Δ				
Δ				
Δ				
Δ				
Δ				
MARK	NATURE OF REVISION	DATE	ENG.	DWN.

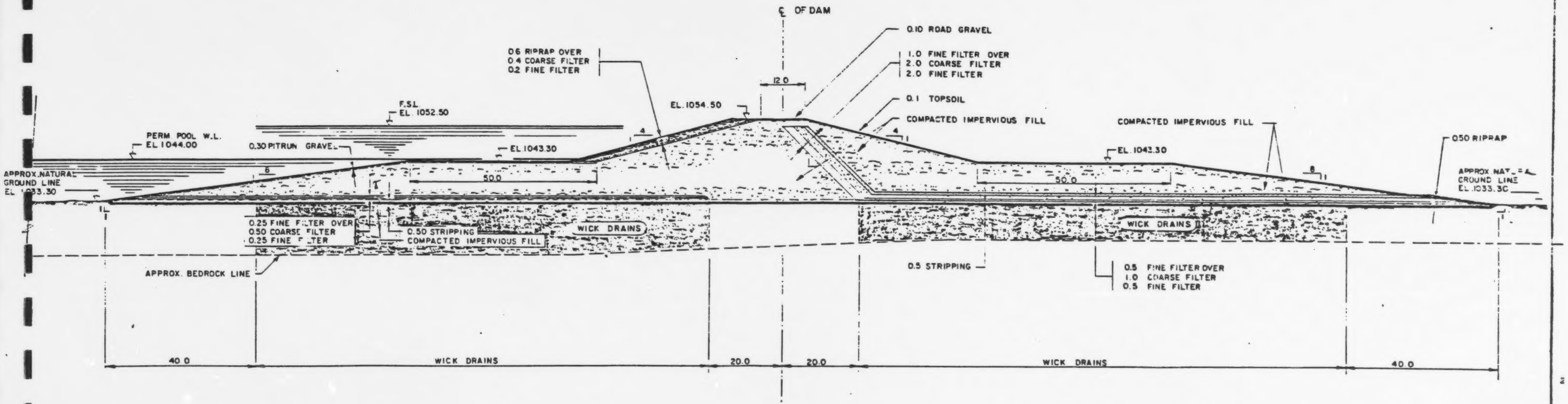
Alberta ENVIRONMENT

SUBMITTED ....
DATE ....
APPROVED ....
DATE ....
DESIGNED ....
CHECKED ....
DRAWN ....
CHECKED I.W.

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

DAM EMBANKMENT GENERAL PLAN

SCALE 1:2500
SHEET 30 OF
DATE SEPT. 19
DWG No. PIC


℄ OF DAM
0.10 ROAD GRAVEL
1.0 FINE FILTER OVER
2.0 COARSE FILTER
2.0 FINE FILTER
0.1 TOPSOIL
COMPACTED IMPERVIOUS FILL
0.6 RIPRAP OVER
0.4 COARSE FILTER
0.2 FINE FILTER
12.0
EL. 1054.50
F.S.L.
EL. 1052.50
PERM. POOL W.L.
EL. 1044.00
0.30 PITRUN GRAVEL
EL. 1043.30
4
1
6
1
50.0
APPROX. NATURAL
GROUND LINE
EL. 1033.30
0.25 FINE FILTER OVER
0.50 COARSE FILTER
0.25 FINE FILTER
0.50 STRIPPING
COMPACTED IMPERVIOUS FILL
WICK DRAINS
APPROX. BEDROCK LINE
COMPACTED IMPERVIOUS FILL
0.50 RIPRAP
EL. 1043.30
4
1
8
1
50.0
APPROX. NATURAL
GROUND LINE
EL. 1033.30
WICK DRAINS
0.5 STRIPPING
0.5 FINE FILTER OVER
1.0 COARSE FILTER
0.5 FINE FILTER
40.0
WICK DRAINS
20.0
20.0
WICK DRAINS
40.0

TYPICAL CROSS SECTION

MARK	NATURE OF REVISION	DATE	ENG	DWN
a				
b				
c				
d				
e				

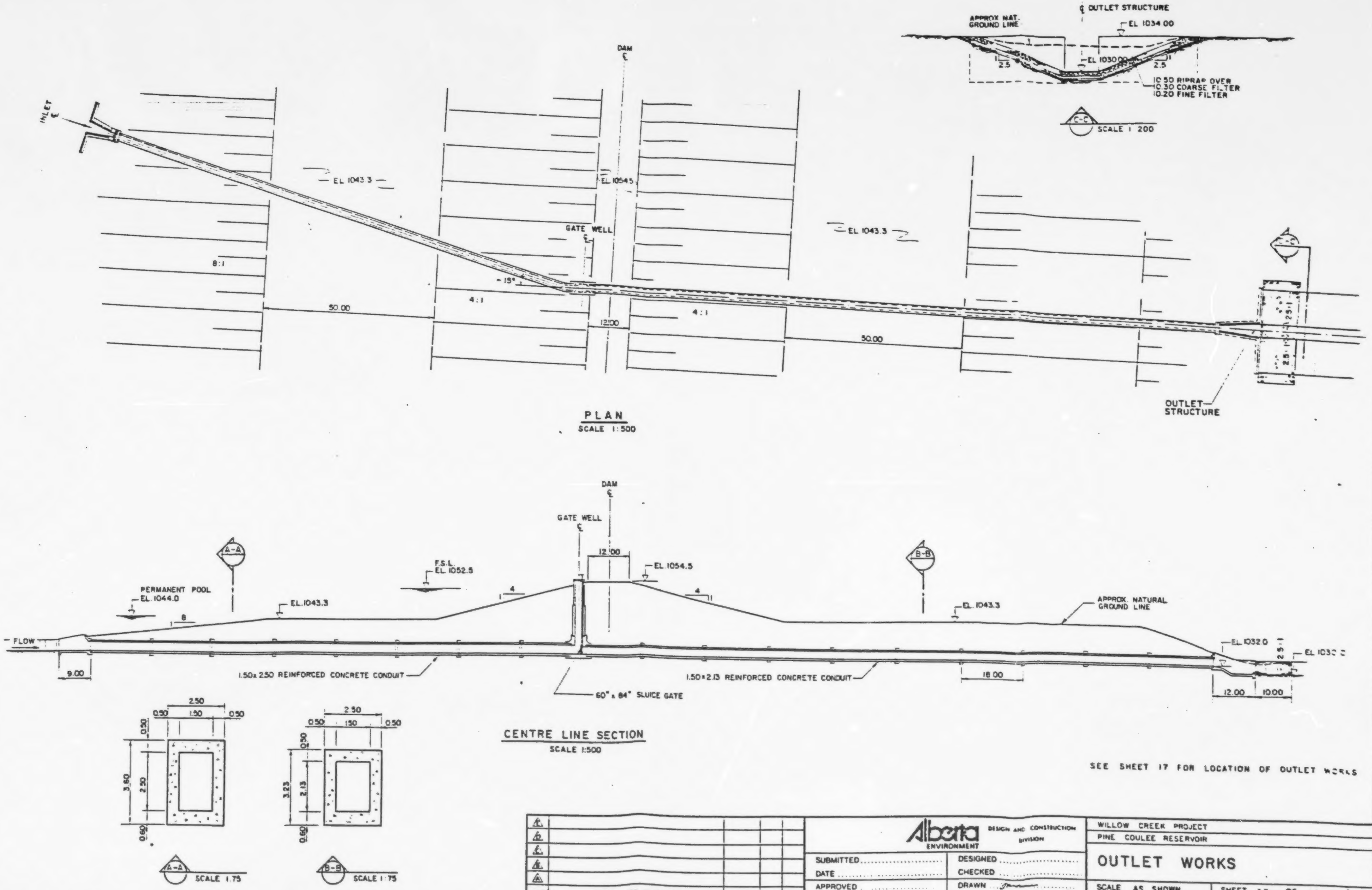
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE ....
DESIGNED ....
CHECKED ....
DRAWN ... F. [illegible] ....
CHECKED ....

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

TYPICAL DAM EMBANKMENT CROSS SECTION

SCALE 1 : 500
DATE AUGUST, 1985
SHEET 31 OF 43
DWG No. PICOR - E85 - 16


℄ OUTLET STRUCTURE
APPROX NAT. GROUND LINE
EL 1034.00
EL 1030.00
1
2.5
1
2.5
1
0.50 RIPRAP OVER
0.30 COARSE FILTER
0.20 FINE FILTER
C-C
SCALE 1:200
DAM ℄
INLET ℄
EL 1043.3
EL 1054.5
GATE WELL ℄
EL 1043.3
8:1
~15°
4:1
4:1
50.00
12.00
50.00
2.5:1
2.5:1
2.5:1
C-C
OUTLET STRUCTURE
DAM ℄
GATE WELL ℄
12.00
EL 1054.5
A-A
B-B
F.S.L. EL 1052.5
PERMANENT POOL EL 1044.0
EL 1043.3
8
4
4
EL 1043.3
APPROX NATURAL GROUND LINE
FLOW
EL 1032.0
EL 1030.0
2.5:1
9.00
1.50 x 2.50 REINFORCED CONCRETE CONDUIT
60" x 84" SLUICE GATE
1.50 x 2.13 REINFORCED CONCRETE CONDUIT
18.00
12.00
10.00
2.50
0.50
1.50
0.50
0.50
3.60
2.50
0.60
A-A
SCALE 1:75
2.50
0.50
1.50
0.50
0.50
3.23
2.13
0.60
B-B
SCALE 1:75

PLAN
SCALE 1:500

CENTRE LINE SECTION
SCALE 1:500

SEE SHEET 17 FOR LOCATION OF OUTLET WORKS

MARK	NATURE OF REVISION	DATE	ENG	DWN
A				
B				
C				
D				
E				

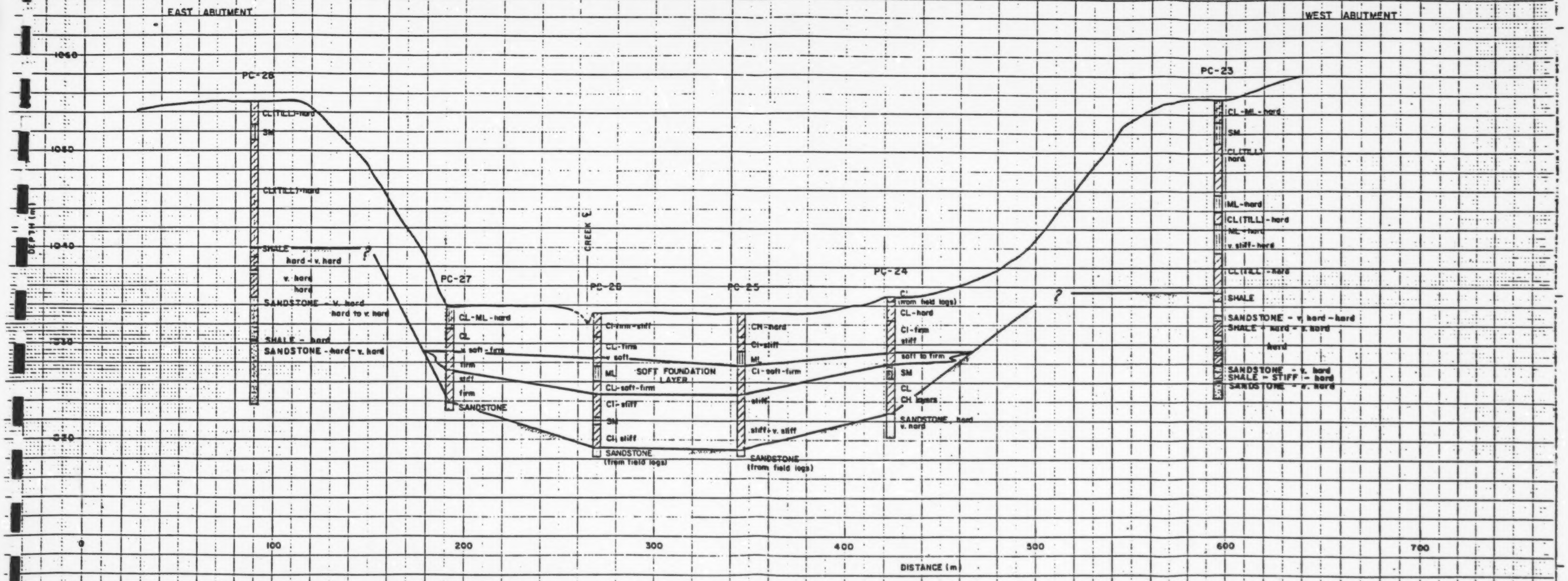
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE
DESIGNED ....
CHECKED ....
DRAWN ....
CHECKED 19.w.

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

OUTLET WORKS

SCALE AS SHOWN
DATE AUGUST, 1985
SHEET 32 OF 43
DWG No PICOR - E85-17


EAST ABUTMENT
WEST ABUTMENT
DEPTH (m)
1060
1050
1040
1030
1020
PC-26
CL(TILL)-hard
SM
CL(TILL)-hard
SHALE
?
hard - v. hard
v. hard
hard
SANDSTONE - v. hard
hard to v. hard
SHALE - hard
SANDSTONE - hard - v. hard
PC-27
CL-ML-hard
CL
v. soft - firm
firm
stiff
firm
SANDSTONE
CREEK
PC-28
CI-firm-stiff
CL-firm
v soft
ML
SOFT FOUNDATION LAYER
CL-soft-firm
CI-stiff
SM
CI, stiff
SANDSTONE
(from field logs)
PC-25
CH-hard
CI-stiff
ML
CI-soft-firm
stiff
stiff - v. stiff
SANDSTONE
(from field logs)
PC-24
CI
(from field logs)
CL-hard
CI-firm
stiff
soft to firm
SM
CL
CH layers
SANDSTONE, hard
v. hard
?
PC-23
CL-ML-hard
SM
CL(TILL)
hard
ML-hard
CL(TILL)-hard
ML-hard
v. stiff-hard
CL(TILL)-hard
SHALE
SANDSTONE - v. hard - hard
SHALE - hard - v. hard
hard
SANDSTONE - v. hard
SHALE - STIFF - hard
SANDSTONE - v. hard
0
100
200
300
400
500
600
700
DISTANCE (m)

MARK	NATURE OF REVISION	DATE	ENG.	DWN
A				
B				
C				
D				
E				

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE ....
DESIGNED ....
CHECKED ....
DRAWN
CHECKED

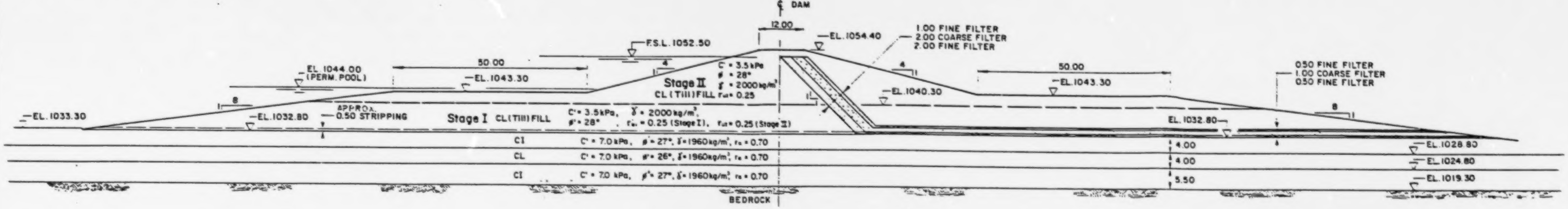
WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

DAM EMBANKMENT
STRATIGRAPHIC SECTION

SCALE AS SHOWN
SHEET 33 OF 43
DATE SEPT. 1985
DWG No PICOR-E85-21

E D C B A

MICROFILM DATE



℄ DAM
12.00
F.S.L. 1052.50
EL. 1054.40
1.00 FINE FILTER
2.00 COARSE FILTER
2.00 FINE FILTER
EL. 1044.00 (PERM. POOL)
50.00
EL. 1043.30
4
1
Stage II
C' = 3.5 kPa
ø = 28°
γ = 2000 kg/m³
CL (Till) FILL r_u = 0.25
EL. 1040.30
4
1
50.00
EL. 1043.30
0.50 FINE FILTER
1.00 COARSE FILTER
0.50 FINE FILTER
8
1
8
1
EL. 1033.30
EL. 1032.80
APPROX. 0.50 STRIPPING
Stage I CL(Till) FILL
C' = 3.5 kPa, γ = 2000 kg/m³,
ø' = 28°, r_u = 0.25 (Stage I), r_u = 0.25 (Stage II)
EL. 1032.80
CI C' = 7.0 kPa, ø' = 27°, γ = 1960 kg/m³, r_u = 0.70
CL C' = 7.0 kPa, ø' = 26°, γ = 1960 kg/m³, r_u = 0.70
CI C' = 7.0 kPa, ø' = 27°, γ = 1960 kg/m³, r_u = 0.70
4.00
4.00
5.50
EL. 1028.80
EL. 1024.80
EL. 1019.30
BEDROCK

DRAWING FILE No PICOR 2[illegible]

CENTRAL FILE No

MARK	NATURE OF REVISION	DATE	ENG	DWN
1				
2				
3				
4				
5				

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

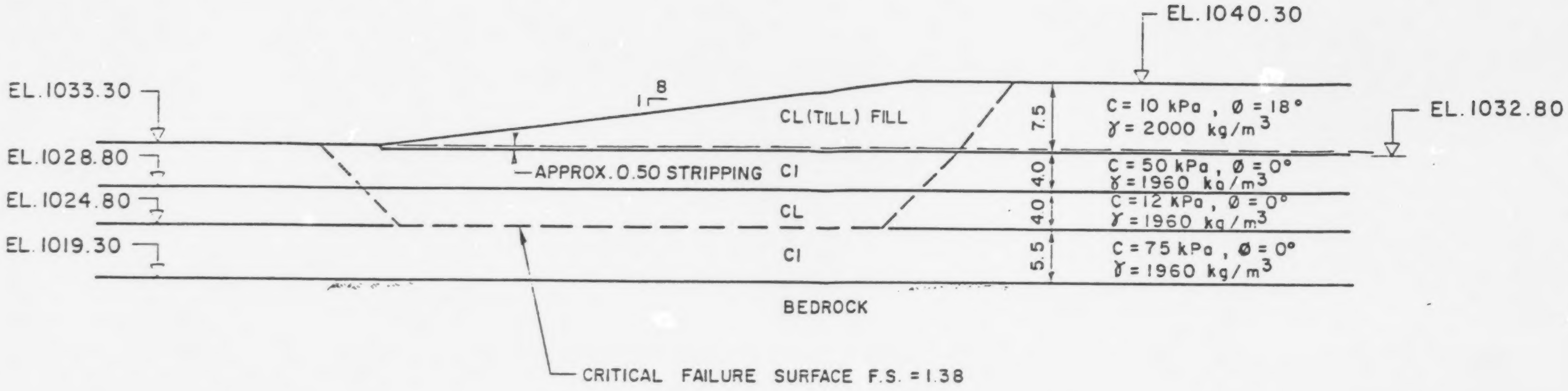
SUBMITTED ....
DATE ....
APPROVED ....
DATE ....
DESIGNED ....
CHECKED ....
DRAWN Tuarlo
CHECKED /

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
DAM EMBANKMENT

CROSS SECTION and STRENGTH PARAMETERS USED for EFFECTIVE STRESS ANALYSIS

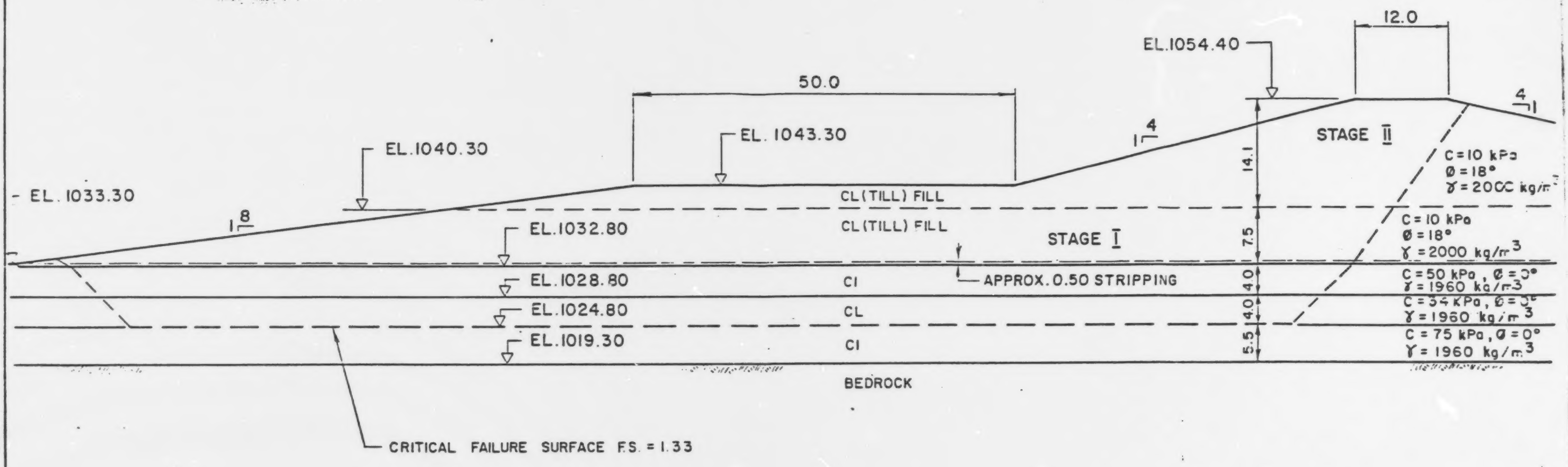
SCALE 1 : 500
SHEET 34 OF 43
DATE SEPTEMBER, 1985
DWG No PICOR - E85-26

B
A


EL. 1040.30
EL. 1033.30
1
8
CL(TILL) FILL
7.5
C = 10 kPa , Ø = 18°
γ = 2000 kg/m³
EL. 1032.80
EL. 1028.80
APPROX. 0.50 STRIPPING
CI
4.0
C = 50 kPa , Ø = 0°
γ = 1960 kg/m³
EL. 1024.80
CL
4.0
C = 12 kPa , Ø = 0°
γ = 1960 kg/m³
EL. 1019.30
CI
5.5
C = 75 kPa , Ø = 0°
γ = 1960 kg/m³
BEDROCK
CRITICAL FAILURE SURFACE F.S. = 1.38

CROSS SECTION AND STRENGTH PARAMETERS USED FOR ANALYSIS IMMEDIATELY AFTER CONSTRUCTION

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION		WILLOW CREEK PROJECT PINE COULEE RESERVOIR
SUBMITTED	DESIGNED	DAM EMBANKMENT (STAGE I) SLOPE STABILITY ANALYSIS TOTAL STRESS
DATE	CHECKED	
APPROVED	DRAWN	SCALE 1 : 500 SHEET 35 OF 43
DATE	CHECKED	DATE SEPT , 1985 DRAWING No PICOR-E85

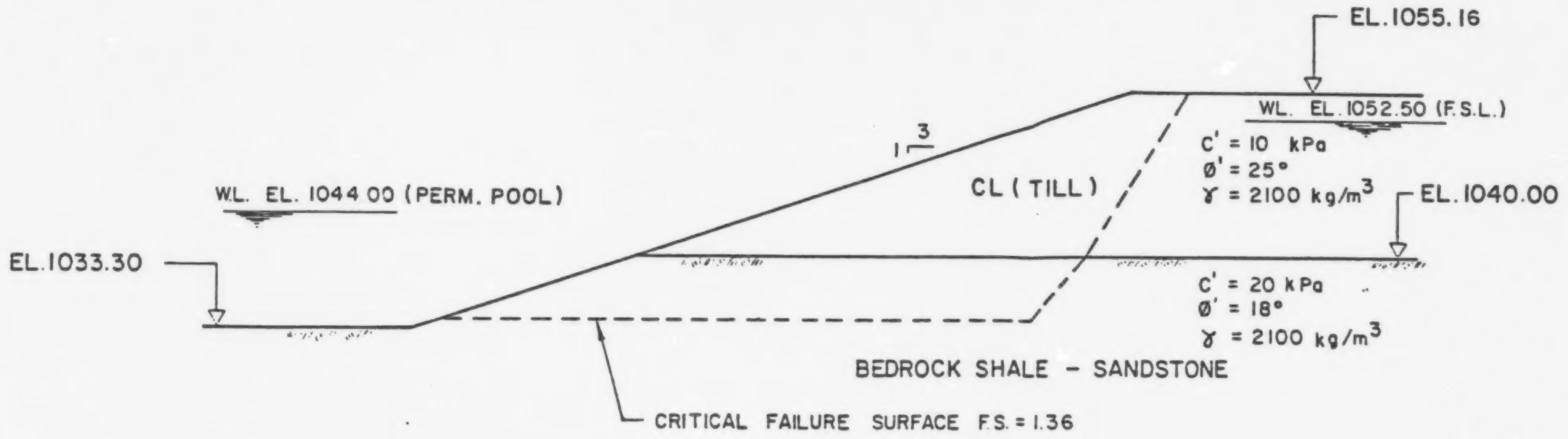

12.0
EL.1054.40
50.0
EL. 1043.30
EL.1040.30
- EL. 1033.30
1 8
1 4
4 1
14.1
7.5
4.0
4.0
5.5
STAGE II
STAGE I
C = 10 kPa
Ø = 18°
γ = 2000 kg/m³
C = 10 kPa
Ø = 18°
γ = 2000 kg/m³
C = 50 kPa , Ø = 0°
γ = 1960 kg/m³
C = 54 kPa , Ø = 0°
γ = 1960 kg/m³
C = 75 kPa , Ø = 0°
γ = 1960 kg/m³
CL (TILL) FILL
CL (TILL) FILL
EL.1032.80
EL.1028.80
EL.1024.80
EL.1019.30
CI
CL
CI
BEDROCK
APPROX. 0.50 STRIPPING
CRITICAL FAILURE SURFACE F.S. = 1.33

CROSS SECTION AND STRENGTH PARAMETERS USED FOR ANALYSIS IMMEDIATELY AFTER CONSTRUCTION CONDITION

Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

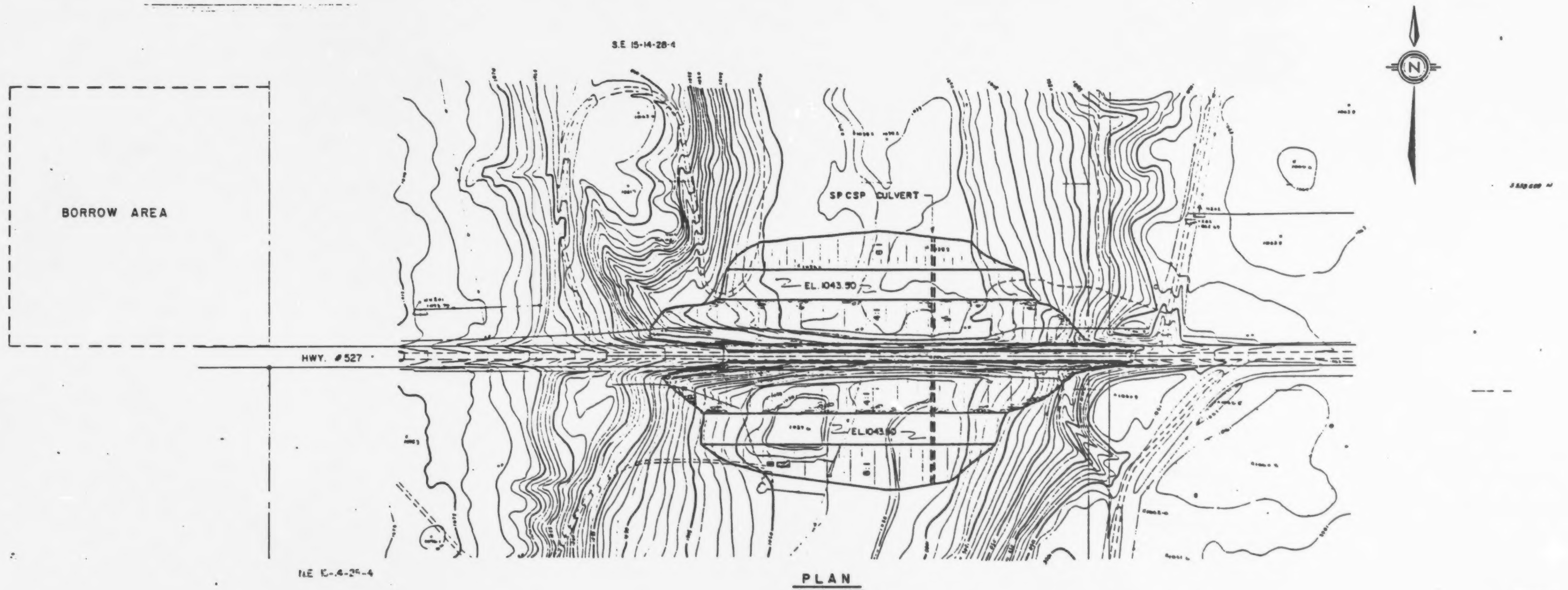
SUBMITTED	DESIGNED
DATE	CHECKED
APPROVED	DRAWN
DATE	CHECKED

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
DAM EMBANKMENT (STAGE II)
SLOPE STABILITY ANALYSIS
TOTAL STRESS
SCALE 1 : 500 SHEET 36 OF 43
DATE SEPT. 1985 DRAWING No. PICOR-E85-3


EL. 1055.16
WL. EL. 1052.50 (F.S.L.)
1
3
$C' = 10$ kPa
$Ø' = 25°$
$\gamma = 2100 \text{ kg/m}^3$
CL (TILL)
WL. EL. 1044.00 (PERM. POOL)
EL. 1040.00
EL. 1033.30
$C' = 20$ kPa
$Ø' = 18°$
$\gamma = 2100 \text{ kg/m}^3$
BEDROCK SHALE - SANDSTONE
CRITICAL FAILURE SURFACE F.S. = 1.36

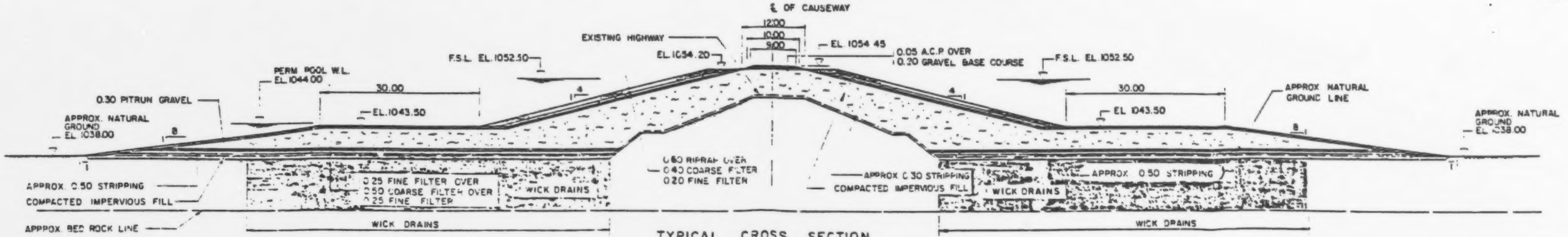
CROSS SECTION AND STRATIGRAPHY REPRESENTS THE EAST BANK, APPROX. 100 m UPSTREAM OF DAM CENTRELINE

Alberta ENVIRONMENT	DESIGN AND CONSTRUCTION DIVISION	WILLOW CREEK PROJECT
		PINE COULEE RESERVOIR
SUBMITTED	DESIGNED	EXISTING RESERVOIR BANK SLOPE STABILITY ANALYSIS RESERVOIR DRAWDOWN
DATE	CHECKED	
APPROVED	DRAWN F. F.	SCALE 1 500 SHEET 37 OF 43
DATE	CHECKED	DATE SEPT. 1985 DRAWING No: PICOR-E85-27


S.E. 15-14-28-4
BORROW AREA
SPCSP CULVERT
EL. 1043.50
EL. 1043.50
HWY. #527
N
N.E. 10-14-28-4

PLAN

SCALE 1:2000


℄ OF CAUSEWAY
12.00
10.00
9.00
EXISTING HIGHWAY
EL. 1054.20
EL. 1054.45
0.05 A.C.P. OVER
0.20 GRAVEL BASE COURSE
F.S.L. EL. 1052.50
F.S.L. EL. 1052.50
PERM POOL W.L.
EL. 1044.00
30.00
30.00
4
4
0.30 PITRUN GRAVEL
EL. 1043.50
EL. 1043.50
APPROX. NATURAL GROUND LINE
APPROX. NATURAL GROUND
EL. 1038.00
APPROX. NATURAL GROUND
EL. 1038.00
8
8
0.60 RIPRAP OVER
0.40 COARSE FILTER
0.20 FINE FILTER
APPROX. 0.50 STRIPPING
COMPACTED IMPERVIOUS FILL
APPROX. BED ROCK LINE
0.25 FINE FILTER OVER
0.50 COARSE FILTER OVER
0.25 FINE FILTER
WICK DRAINS
APPROX. 0.30 STRIPPING
COMPACTED IMPERVIOUS FILL
WICK DRAINS
APPROX. 0.50 STRIPPING
WICK DRAINS
WICK DRAINS

TYPICAL CROSS SECTION

SCALE 1:400

MARK	NATURE OF REVISION	DATE	ENG.	DWN.
1				
2				
3				
4				
5				

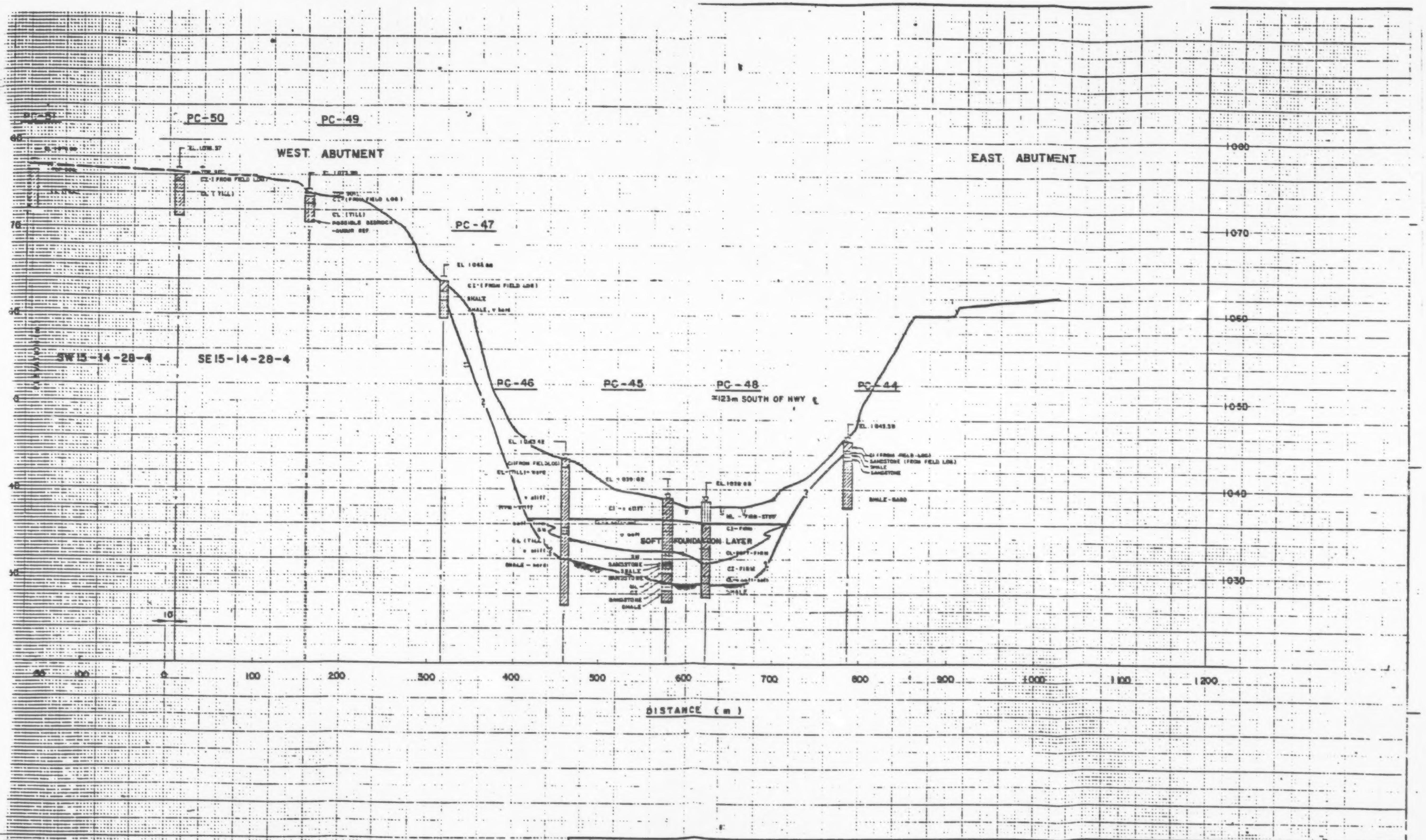
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED
DATE
APPROVED
DATE
DESIGNED
CHECKED
DRAWN
CHECKED G. W.

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

CAUSEWAY @ HIGHWAY # 527

SCALE AS SHOWN
SHEET 38 OF 43
DATE AUGUST, 1985
DWG. No. PICOR - E85 - 9


PC-51
PC-50
PC-49
WEST ABUTMENT
EAST ABUTMENT
PC-47
EL. 1076.37
CI-I (FROM FIELD LOG)
CL T (TILL)
EL. 1073.80
TOP SOIL
CI (FROM FIELD LOG)
CL (TILL)
POSSIBLE BEDROCK
EL. 1061.66
CI (FROM FIELD LOG)
SHALE
SHALE, v hard
SW 15-14-28-4
SE 15-14-28-4
PC-46
PC-45
PC-48
≈123m SOUTH OF HWY ℄
PC-44
EL. 1043.42
CI (FROM FIELD LOG)
CL-(TILL)- hard
EL. 1039.62
EL. 1038.68
EL. 1043.38
CI (FROM FIELD LOG)
SANDSTONE (FROM FIELD LOG)
SHALE
SANDSTONE
SHALE - HARD
v stiff
CI - v stiff
ML - FIRM-STIFF
CI - FIRM
v soft
SOFT FOUNDATION LAYER
CL (TILL)
v stiff
SHALE - hard
SW
SANDSTONE
SHALE
SANDSTONE
GL
CI
SANDSTONE
SHALE
CL - SOFT-FIRM
CI - FIRM
SHALE
1080
1070
1060
1050
1040
1030
ELEVATION (m)
10
100
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
DISTANCE (m)

MARK	NATURE OF REVISION	DATE	ENG	DWN
1				
2				
3				
4				
5				

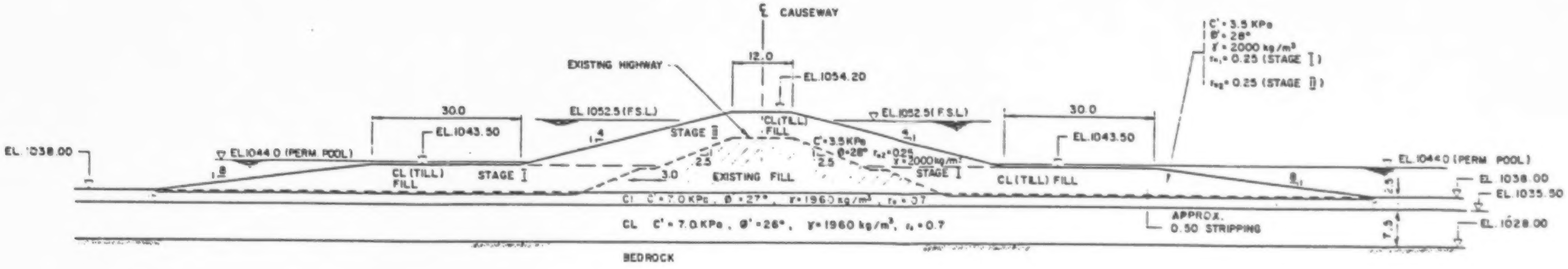
Alberta ENVIRONMENT
DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE ....
DESIGNED ....
CHECKED ....
DRAWN ....
CHECKED ....

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

STRATIGRAPHIC SECTION
@ 40 m NORTH OF HWY # 527

SCALE HORIZ 2000 VERT 500
SHEET 39 OF 43
DATE SEPTEMBER, 1985
DWG No. [illegible]-22


℄ CAUSEWAY
12.0
EXISTING HIGHWAY
EL.1054.20
30.0
EL 1052.5 (F.S.L.)
EL.1052.5 (F.S.L.)
30.0
C' = 3.5 KPa
Ø' = 28°
γ = 2000 kg/m³
r_{u1} = 0.25 (STAGE I)
r_{u2} = 0.25 (STAGE II)
EL. 1038.00
EL.1044.0 (PERM. POOL)
EL.1043.50
1 8
1 4
STAGE II
CL(TILL) FILL
CL (TILL) FILL
STAGE I
2.5
3.0
EXISTING FILL
C' = 3.5 KPa
Ø = 28° r_{u2} = 0.25
γ = 2000 kg/m³
2.5
STAGE I
4 1
EL.1043.50
CL (TILL) FILL
8 1
EL.1044.0 (PERM. POOL)
EL. 1038.00
EL. 1035.50
EL. 1028.00
2.5
7.5
CL C' = 7.0 KPa, Ø' = 27°, γ = 1960 kg/m³, r_u = 0.7
CL C' = 7.0 KPa, Ø' = 26°, γ = 1960 kg/m³, r_u = 0.7
APPROX. 0.50 STRIPPING
BEDROCK

MARK	NATURE OF REVISION	DATE	ENG	DWN
E				
D				
C				
B				
A				

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

SUBMITTED ....
DATE ....
APPROVED ....
DATE
DESIGNED ....
CHECKED ....
DRAWN
CHECKED

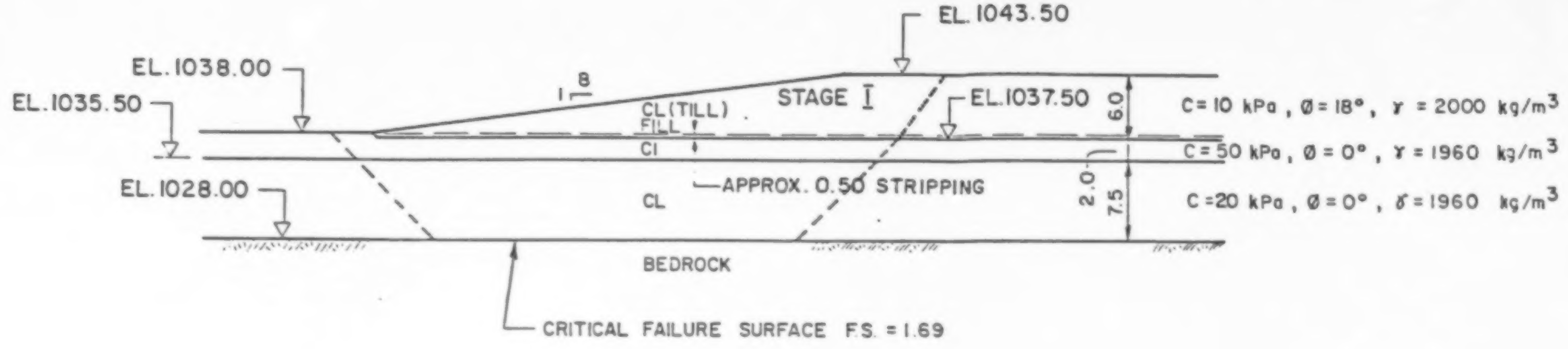
WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
CAUSEWAY ℄ HWY # 527

CROSS SECTION & STRENGTH PARAMETERS USED FOR EFFECTIVE STRESS ANALYSIS

SCALE 1 500
SHEET 40 OF 43
DATE SEPT. 1985
DWG. No. PICOR - E85 - 25

-560 I 160 R-5

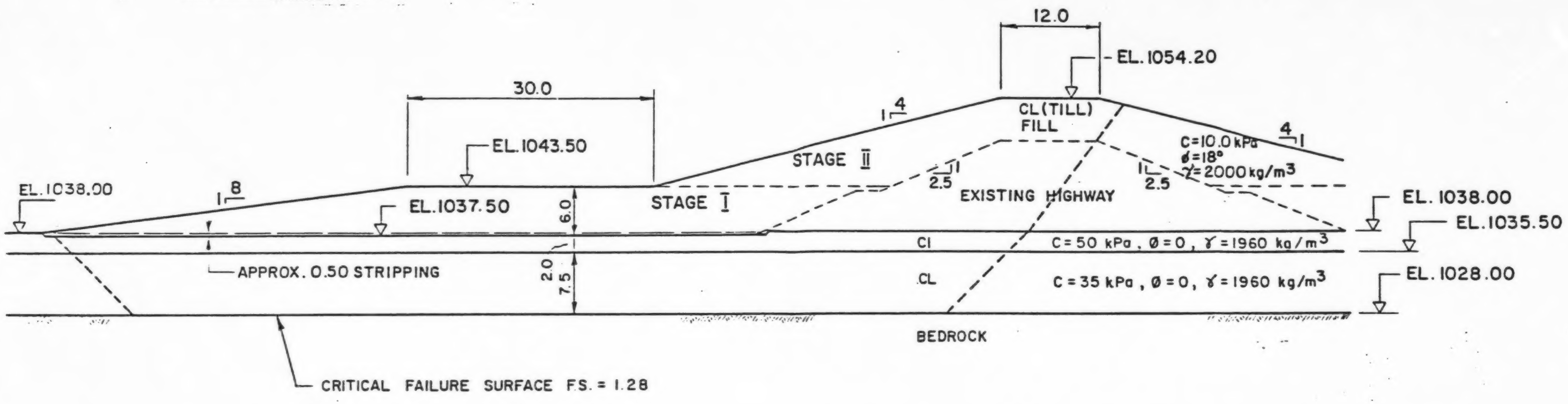
0
10
20
40
60
80
100


EL. 1043.50
EL. 1038.00
EL. 1035.50
1
8
CL (TILL)
STAGE I
EL. 1037.50
6.0
FILL
CI
APPROX. 0.50 STRIPPING
EL. 1028.00
CL
2.0
7.5
BEDROCK
CRITICAL FAILURE SURFACE F.S. = 1.69
C = 10 kPa , Ø = 18°, γ = 2000 kg/m³
C = 50 kPa , Ø = 0°, γ = 1960 kg/m³
C = 20 kPa , Ø = 0°, δ = 1960 kg/m³

CROSS SECTION AND STRENGTH PARAMETERS USED FOR ANALYSIS IMMEDIATELY AFTER CONSTRUCTION CONDITION.

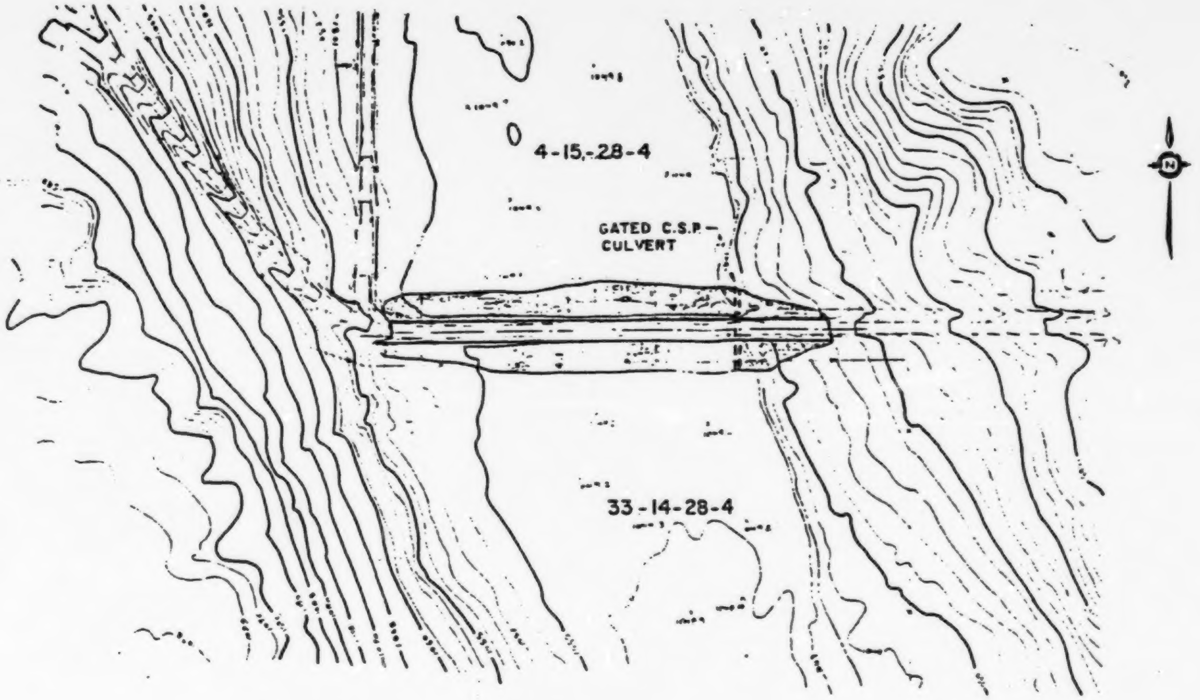
Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION
SUBMITTED
DATE
APPROVED
DATE
DESIGNED
CHECKED
DRAWN
CHECKED

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR
CAUSEWAY @ HIGHWAY # 527
SLOPE STABILITY ANALYSIS (STAGE I)
TOTAL STRESS
SCALE 1 : 500
SHEET 41 OF 43
DATE
PICOR-E89-31


12.0
EL. 1054.20
30.0
1 4
CL(TILL)
FILL
4 1
C=10.0 kPa
ϕ=18°
γ=2000 kg/m³
EL. 1043.50
STAGE II
1 2.5
1 2.5
EL. 1038.00
1 8
STAGE I
EXISTING HIGHWAY
EL. 1037.50
6.0
EL. 1038.00
EL. 1035.50
CI
C = 50 kPa , Ø = 0 , γ = 1960 kg/m³
APPROX. 0.50 STRIPPING
2.0
7.5
CL
C = 35 kPa , Ø = 0 , γ = 1960 kg/m³
EL. 1028.00
BEDROCK
CRITICAL FAILURE SURFACE F.S. = 1.28

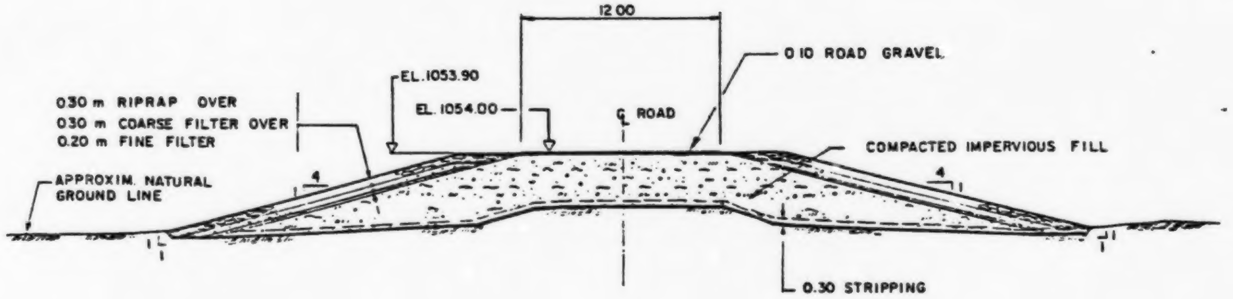
CROSS SECTION AND STRENGTH PARAMETERS USED FOR ANALYSIS IMMEDIATELY AFTER CONSTRUCTION CONDITION

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION		WILLOW CREEK PROJECT
		PINE COULEE RESERVOIR
SUBMITTED	DESIGNED	CAUSEWAY @ HIGHWAY # 527 SLOPE STABILITY ANALYSIS (STAGE II) TOTAL STRESS
DATE	CHECKED	
APPROVED	DRAWN	SCALE 1 : 500 SHEET 42 OF 43
DATE	CHECKED /	DATE DRAWING No PICOR-E85-28


4-15,-28-4
GATED C.S.P. CULVERT
33-14-28-4
N

PLAN

1:2 000


12.00
0.10 ROAD GRAVEL
EL. 1053.90
EL. 1054.00
℄ ROAD
0.30 m RIPRAP OVER
0.30 m COARSE FILTER OVER
0.20 m FINE FILTER
COMPACTED IMPERVIOUS FILL
APPROXIM. NATURAL GROUND LINE
4
1
0.30 STRIPPING

TYPICAL CROSS SECTION

1:200

MARK	NATURE OF REVISION	DATE	ENG	DWN
E				
D				
C				
B				
A				

Alberta ENVIRONMENT DESIGN AND CONSTRUCTION DIVISION

SUBMITTED	DESIGNED
DATE	CHECKED
APPROVED	DRAWN
DATE	CHECKED

WILLOW CREEK PROJECT
PINE COULEE RESERVOIR

CAUSEWAY E.-W. ROAD

N. BDY. OF 33-14-28-W4

SCALE AS SHOWN | SHEET 43 OF 43
DATE AUGUST, 1985 | DWG. No. PICOR-E85-20

POOR

QUALITY

ORIGINAL